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PAPERS, REPORTS, DISCUSSIONS, AND MEMOIRS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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EXPERIMENTS TO DETERMINE RATE OF EVAPORATION FROM SATURATED SOILS AND RIVER-BED SANDS

By Ralph L. Parshall,* Assoc. M. Am. Soc. C. E.

Synopsis

Much study has already been given by various investigators to the loss of moisture from soils by evaporation, as well as the effect of mulches in retarding or preventing this loss.† The moisture content of the soil, which is held in capillary form, is least at the surface where the evaporation occurs, and leaves there a deposit of such salts as were dissolved in the soil moisture. Heavy Western soils, which usually carry alkaline salts, have the greatest capillary attraction; that is, such soils will bring moisture to the surface from greater depths but at a slower rate than the sandy or the lighter soils.

This paper is based on a study of the evaporation loss from soils contained in tanks under like exposure, the soils ranging from coarse river sand to heavy dark alkaline soil, with the water-table maintained at 1, 6, and 12 in. beneath the surface. It is not the writer's intention to present conclusive data as to the loss by evaporation from moist soil surfaces, but rather to describe the methods and procedure followed in this study and submit such information as is now available as an indication of the general trend of the relations existing between the various soils and conditions as well as supporting the general conclusions drawn.

Description of Apparatus

Heretofore, the evaporation loss from soils has been determined by weighing the soil mass contained in tanks at stated intervals, the difference in weights

Note.—The Special Committee on Irrigation Hydraulics has selected the subject of "Evaporation from Soils" as one of ten for study and research. This paper was submitted to the Committee by its author, and the Committee has recommended its publication in *Proceedings* in order to elicit discussion of the subject (see Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 173). Written discussion on this paper will be closed in August, 1929, *Proceedings*.

* Senior Irrig. Engr., U. S. Dept. of Agriculture, Fort Collins, Colo.

† Progress Report of the Special Committee on Irrigation Hydraulics, *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 177.

equalling the loss by evaporation for the period. The problem of maintaining a constant and fixed position of the water-table in the soil, as well as that of determining the rate of loss due to evaporation, was solved, in the experiments herein reported, by the use of the principle of the Mariotte flask. In this regulating apparatus (Fig. 1) a calibrated air-tight drum, *D*, serves as the supply reservoir and is connected to a buried soil tank by a ½-in. supply pipe. A glass tube is mounted beside the supply reservoir and is connected to the system at both top and bottom so that the water level in the drum may be observed. The quantity withdrawn is indicated by differences read on the meter stick.

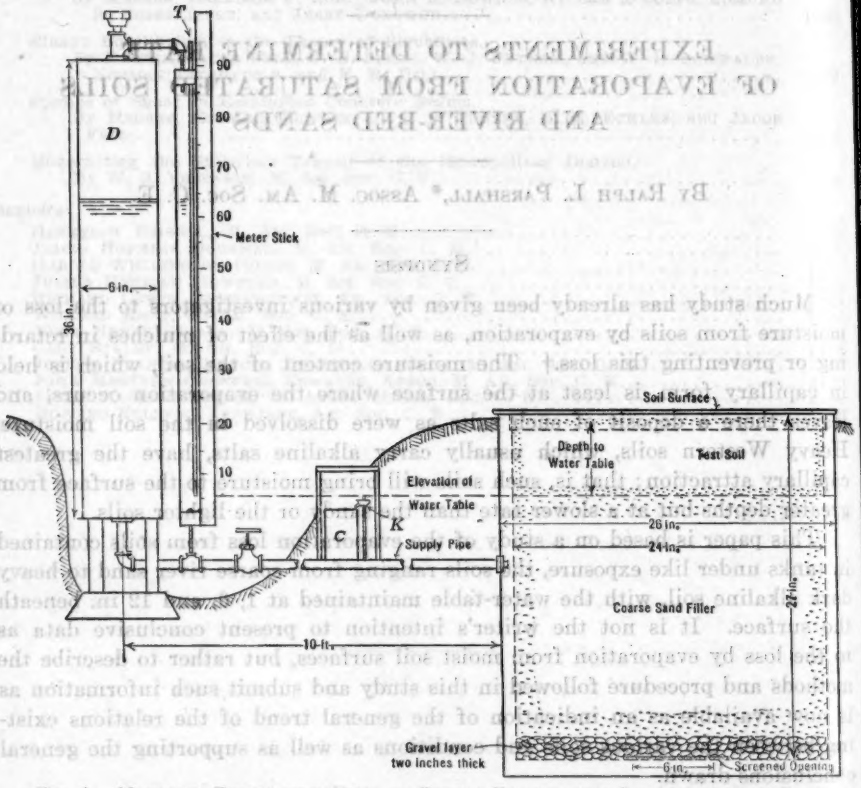

FIG. 1.—MARIOTTE REGULATION FEED TO SUPPLY EVAPORATION LOSS FROM SOIL TANKS.

Previous experience with the apparatus indicated that variations of temperature of the partial vacuum in the supply reservoir changed the vapor pressure to such an extent that dependable readings were not possible for short periods. In order to reduce the effects of outside temperature changes to a minimum all supply reservoirs are well covered with corrugated asbestos board next to the drum, *D*, and this is covered with one layer of building paper. Each individual set of apparatus is mounted in a wooden housing, and the space between the insulated drum and the sides of the housing is filled with clean dry wood shavings. A hinged lid at the top of the housing is provided for convenience in refilling the supply reservoir.

The valve in the supply pipe is necessary to prevent over-charge of the soil tank when the supply reservoir is being refilled. To insure against the water-table in the soil tank being overcharged, either accidentally or by excess water entering from the supply reservoir as an effect of temperature, a vertical tube, *K* (Fig. 1), is attached to the supply pipe. The top end agrees in elevation with the plane of the water-table in the soil tank. Excess water in this tank is wasted through this small tube into the container, *C*, where it is measured, and the indicated evaporation loss corrected accordingly.

The outside of the housing is whitewashed. A bulb of a thermograph has been sealed inside the supply reservoir of Tank No. 17 (Fig. 2) to indicate the variation of temperature. Covers are provided for all tanks to shield them against rain.

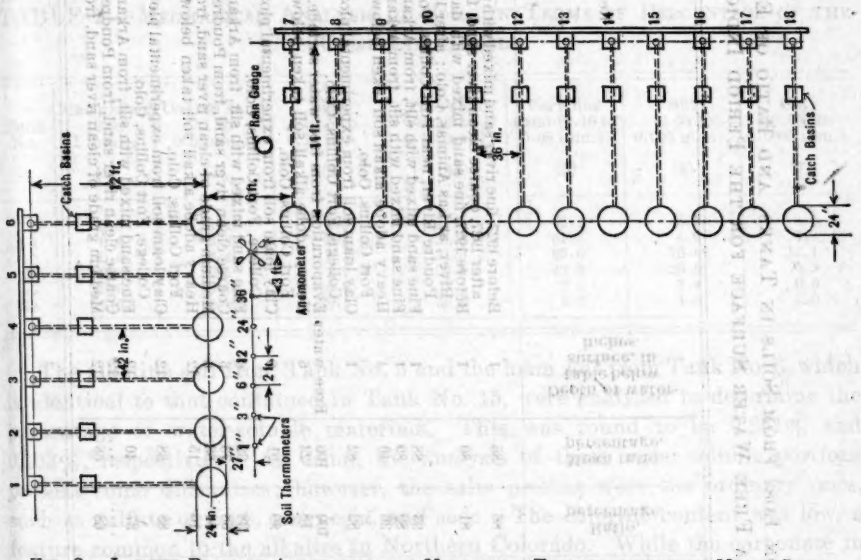

FIG. 2.—LAYOUT PLAN OF APPARATUS FOR MEASURING EVAPORATION FROM MOIST SOILS.

The soil tank consists of two shells, the outer constituting a water-tight container, while the inner, containing the soil, is supported at the top by an angle-iron rim so that a 1-in. space separates the inner tank from the outer at the sides and bottom. A hole, 6 in. in diameter, through the bottom of the inner tank, is covered by a coarse screen, and immediately above this is a 2-in. layer of gravel. Except in two tanks, coarse sand is placed on the gravel layer up to the plane of the water-table, and on this porous material the test soil is placed. The water contained between the inner and outer shells of the tank is fed upward through the large hole in the bottom of the inner tank and thence through the soil to supply the moisture lost by evaporation at the surface. Loss of water by evaporation from the water surface between the two shells is prevented by sealing the top of the outer tank with heavy canvas and coal-tar. Tanks Nos. 1 to 6, inclusive, contain different types of soil, with the water-table at 1 in. below the soil surface. Tanks Nos. 7 and 8 contain water only. In Tanks Nos. 9 to 13, inclusive, the water-table is 6 in. beneath the surface,

TABLE 1.—AVERAGE EVAPORATION LOSS FROM SOILS IN TANKS, AND RATIO OF EVAPORATION FROM SOILS TO THAT FROM A FREE WATER SURFACE FOR THE PERIOD INDICATED.

Tank No.	Loss, September 8 to 8, 1925, in inches.	Ratio, percentage.	Loss, September 8 to 18, 1925, in inches.	Ratio, percentage.	Loss, September 24 to 30, 1925, in inches.	Ratio, percentage.	Mean ratio, percentage.	Depth of water-table below surface, in inches.	Remarks.
1	0.172	85	0.180	85	0.144	86	85	1	Before 1927, fine river sand mixed with silt, from Arkansas River, at Lamar, Colo.; after 1927, coarse clean river sand, from Poudre River, near Fort Collins, Colo.
2	0.162	80	0.169	80	0.136	81	80	1	Before 1927, fine sand mixed with dark-colored silt and mud, from Arkansas River, at Las Animas, Colo.; after 1927, mixing grade, clean river sand, from Poudre River, near Fort Collins, Colo.
3	0.171	85	0.182	86	0.156	93	88	1	Fine sand mixed with silt, from Arkansas River, at Avondale, Colo.
4	0.177	88	0.191	90	0.150	89	89	1	Fine sand mixed with silt, from Arkansas River, at Rocky Ford, Colo.
5	0.095	47	0.081	38	0.061	36	40	1	Heavy adobe alkali soil taken below grass roots in marsh, 3 miles south of Fort Collins, Colo.
6	0.166	82	0.173	82	0.137	82	82	1	Clay loam soil from experimental plots on the campus of Colorado Agricultural College, Fort Collins, Colo.
7–8	0.202	100	0.212	100	0.168	100	100	Free water	Evaporation from free water surface, mean value.
9	0.061	30	0.047	22	0.031	18	23	6	Heavy adobe alkali soil taken below grass roots in marsh, 3 miles south of Fort Collins, Colo.
10	0.186	92	0.197	93	0.149	89	91	6	Clay loam soil from experimental plots on the campus of Colorado Agricultural College, Fort Collins, Colo.
11	0.064	32	0.058	27	0.064	38	32	6	Fine sand mixed with silt, from Arkansas River, at Rocky Ford, Colo.
12	0.159	79	0.154	73	0.115	68	73	6	Coarse clean river sand, from Poudre River, at Fort Collins, Colo.
13	0.176	87	0.168	79	0.141	84	83	6	Medium grade of clean river sand, from Poudre River, at Fort Collins, Colo.
14	0.040	20	0.029	14	0.016	10	15	12	Heavy adobe alkali soil taken below grass roots in marsh, 3 miles south of Fort Collins, Colo.
15	0.183	91	0.181	85	0.148	88	88	12	Clay loam soil from experimental plots on the campus of Colorado Agricultural College, Fort Collins, Colo.
16	0.088	44	0.080	38	0.062	37	40	12	Fine sand mixed with silt, from Arkansas River, at Rocky Ford, Colo.
17	.....	....	.....	....	.....	....	....	12	Coarse clean river sand, from Poudre River, at Fort Collins, Colo.
18	0.142	70	0.140	66	0.114	68	68	12	Medium grade of clean river sand, from Poudre River, at Fort Collins, Colo.

while in Tanks Nos. 14 to 18, inclusive, it is 12 in. beneath the surface. Six soil thermometers are placed parallel to the axis of Tanks Nos. 1 to 6 at depths ranging from 1 to 36 in. A Robinson anemometer and a standard rain-gauge are mounted near Tank No. 7. Tanks Nos. 7 and 8 are equipped with hook-gauges, reading to 0.01 in. A maximum thermometer is supported in the former and a minimum thermometer in the latter.

The Arkansas River sand in Tanks Nos. 1 and 2 was removed at the beginning of the 1927 season and coarse and medium river sand substituted. The materials in all other tanks used in 1927 were the same as those used in 1926 (see Table 1). A mechanical analysis of the materials in six of the tanks is shown in Table 2.

TABLE 2.—MECHANICAL ANALYSIS OF SOILS IN TERMS OF PERCENTAGE OF THE COMPLETE SAMPLE.

Tank No.	Fine gravel (2 to 1 mm.). (1)	Coarse sand (1 to 0.5 mm.). (2)	Sand (0.5 to 0.25 mm.). (3)	Fine sand (0.25 to 0.10 mm.). (4)	Very fine sand (0.10 to 0.05 mm.). (5)	Silt (0.05 to 0.005 mm.). (6)	Clay (0.005 to 0.0001 mm.). (7)
3	0.6	1.1	2.0	27.8	62.2	5.7	0.6
4	5.4	3.2	2.3	21.8	61.5	4.9	0.9
5	0.4	4.4	7.2	31.5	33.6	10.8	12.1
15	0.8	2.1	2.9	20.0	42.8	25.5	5.9
17	35.6	37.0	16.8	7.1	1.5	1.4	0.6
18	7.5	20.0	29.5	36.0	5.0	1.0	1.0

The alkaline soil from Tank No. 5 and the loam soil from Tank No. 6, which is identical to that contained in Tank No. 15, were analyzed to determine the percentage of water-soluble materials. This was found to be 0.261% and 0.153%, respectively. As usual, the analysis of these water-soluble portions present some difficulties; however, the salts present were the ordinary ones, such as sulfate of lime, magnesia, and soda. The chlorine content was low, a feature common to the alkalies in Northern Colorado. While the carbonate in Tank No. 5 was about the usual quantity, that in Tank No. 6 appeared to be greater. It seems to be a question as to how much manganese, phosphoric acid, alumina, and ferric oxide may be expected in the water-soluble portion.

All tanks are not filled in the same manner. In Tanks Nos. 1 to 6, the water-table is carried at a nominal depth of 1 in. Tanks Nos. 1 to 4, inclusive, are filled with coarse river sand to within about 4 in. of the top. The sample material, 2½ in. deep, is then added, which leaves about 1½ in. between the top of the soil and the rim of the tank. Tank No. 5, which contains the heavy alkali soil, was filled from the gravel layer in the bottom to a point within about 1½ in. of the rim. Tank No. 6 was also completely filled in a manner similar to Tank No. 5, with soil taken from the excavation made in installing the equipment. Tanks Nos. 9 to 13 were filled with coarse sand to 6 in. below the top or rim, above which was placed the test soil. Tanks Nos. 14 to 18 were likewise filled to a level 12 in. below the rim and topped with the test soil.

In the case of Tanks Nos. 5 and 6, which were completely filled with heavy soil, the intention was to ascertain what relative effect would be apparent

when the water that was lost in evaporation was drawn through a greater depth of soil than that represented in the other tanks.

Tanks Nos. 1 to 6 were the first set installed. The angle-iron rim at the top of the inner tank was placed so that the inner tank was exposed above the ground surface about 1½ in. This arrangement was improved in Tanks Nos. 7 to 18, by banking soil around the tank as shown in Fig. 1.

The surface of the soil in all tanks was kept free of plant growth and in an undisturbed condition. The high water-table in Tanks Nos. 1 to 6 developed a slight surface scum of algæ or bacterial formation. This yellowish-brown coating is thin and more or less elastic. What effect it may have on the rate of evaporation is not known, but it is believed to be small. No doubt, copper sulfate added in small quantities to the surface of the soil would retard or prevent the growth, but because a natural condition was sought, the surface was left untouched.

The appearance and texture of the soil surfaces were observed on September 17, 1926. Tanks Nos. 1 to 6 were all yellowish-brown with a scum of growth about 1/32 in. in thickness. This covering or membrane was somewhat elastic, but did not cover the entire surface; that is, it was in patches of various areas. The edges of these patches were in some cases roller back. The layer of scum on the soil surface of Tank No. 6 was thin. A deposit of alkali showed on the surface of Tank No. 5. The soil surface in Tank No. 9 was a light gray color, showing some alkali. Small cracks appeared here and there, with the soil becoming dry and mealy.

The soil in Tank No. 10 was dark brown, with damp surface and patches of scum. A very thin scum showed on the soil surface of Tank No. 11, the soil being damp, with concentrated isolated alkali crystals, compact and dark colored. No alkali showed on Tanks Nos. 12 and 13. The surface was damp with a thin scum, generally greenish-brown in color.

The surface of Tank No. 14 was almost white with an alkali deposit, very irregular or uneven, dry and mealy, with a few small surface cracks. This surface, when touched, would crumble to a fluffy mass. No alkali appeared on the soil surface of Tank No. 15 which was damp, with a very slight, brownish scum, and of mealy texture. Concentrated alkali crystals were spread over one-half the surface area of Tank No. 16 which was damp, with slight scum; the soil was more compact. The surface of Tank No. 17 was clean, smooth, and dry. Tank No. 18 showed a damp surface, inclined to be slightly incrusted.

Effect of Depth to Water-Table on Rate of Evaporation

Soil Tanks Nos. 1 to 6 were set up and filled in May, 1925, and Tanks Nos. 7 to 18 were installed in July, 1925. Because the filling of soil was uncompacted and the apparatus untried, only preliminary observations were made during this first season to ascertain the characteristics of the equipment. Fragmentary results of the comparison of the evaporation loss from the several moist soils and the free water surface are given in Table 1. This record covers only a part of September, 1925. The insulation for the Mariotte tanks was not sufficient at first to give reliable readings because of the variation of pressure of the water vapor in the space, *D* (Fig. 1). Considerable trouble

was also caused by leakage of air into the Mariotte system. The instability of this equipment and the fact that the tanks were newly filled do not justify the placing of much dependence on these first results, although they show, in a general way, the trend of the evaporation loss as determined for the season of 1926. The period from September 14 to 23, 1925, was excluded because of the interference of rainy weather.

At the beginning of the season of 1926, the Mariotte tanks were completely overhauled and re-set. Thermograph records of the temperature of the partial vacuum within the supply reservoir (Tank No. 17) show a maximum daily range of 7° Fahr. and a minimum range of 1° Fahr. This range of temperature change is sufficient to affect materially the night and morning readings of the water level in the Mariotte supply reservoir. Theoretically, there can be no return of water to the supply reservoir from the soil tank as a result of reduction of the pressure of the water vapor in the space, *D* (Fig. 1). In order to investigate the apparent inconsistency of the action of the Mariotte regulation, special apparatus was set up by which the pressure of the water vapor in the space, *D*, could be greatly increased and decreased by changing the temperature. It was demonstrated that when a very slight decrease in temperature occurred, with no evaporation from the exposed sustaining or balancing water surface, an accumulation of water in the supply reservoir was possible. During a period of falling temperature, the pressure of the water vapor decreases slowly, and for a condition of balanced pressures a small bubble of air is held at the lower end of the small brass vent tube, *T* (Fig. 1). The surface tension holding this bubble in place operates as a seal at the lower end of the vent tube. Manifestly, if this tube is sealed, a decrease of pressure in the space, *D*, will cause water to flow from the soil tank into the reservoir of the Mariotte apparatus.

At first, it was thought possible to differentiate between the night and day losses of evaporation from the several types of soil, but when the data were studied thoroughly, no correlation of these losses appeared. The observations during the season of 1926 were taken at approximately 8:00 A. M., an hour when the air temperature was reasonably uniform for the weekly periods. For the season of 1927, the observations were taken at about 11:00 P. M.

Table 3 gives the average evaporation loss, in inches per day, for 1926 and 1927 and a comparison of losses from the soils with that from a water surface. During 1926, a complete evaporation record from a free-water surface was also observed in an area distant about 500 ft. northwest of the experimental apparatus. This is designated Tank *A* in Table 3.

Covers were provided to protect the tanks against rain, but nearly each week more or less rain fell on the exposed soil surfaces. It was found practically impossible to guard against this condition because all soil surfaces were being exposed to natural conditions. Experience with these tanks shows that a marked reduction in the evaporation loss occurs when the covers are in place.

The cooling effect of the rain on the soil increases the surface tension of the capillary moisture drawn up from the water-table. Rain water falling on the soil also dilutes the soil solution and if the solution is alkaline it

increases the rate of evaporation. It is evident that, although adding moisture to the soil at the time, light showers may later cause a more rapid depletion of the moisture already within the soil. Messrs. F. H. King and J. B. Stewart,* have also noted this phenomenon.

TABLE 3.—AVERAGE EVAPORATION LOSS, IN INCHES PER DAY, FROM MOIST SOILS AND MEAN RATIO OF EVAPORATION FROM SOILS TO THAT FROM A FREE WATER SURFACE TAKEN AS 100.

Tank No.	Depth of water-table below surface, in inches.	JUNE 23 TO OCTOBER 20, 1926.		MAY 1 TO OCTOBER 30, 1927.		Depth of test soil, in inches.
		Evaporation loss, in inches.	Ratio.	Evaporation loss, in inches.	Ratio.	
1	1	0.169	99	0.166	106	3
2	1	0.166	96	0.171	109	3
3	1	0.180	100	0.159	101	3
4	1	0.178	100	0.165	105	3
5	1	0.132	75	0.131	83	Full depth of tank
6	1	0.143	83	0.136	87	Full depth of tank
7 and 8	Free water	0.173*	100	0.157	100	....
9	6	0.048	28	0.105	67	6
10	6	0.176	102	0.154	98	6
11	6	0.126	73	0.147	94	6
12	6	0.171	99	0.104	66	6
13	6	0.184	104	0.158	101	6
14	12	0.019	10	0.051	32	12
15	12	0.097	56	0.129	82	12
16	12	0.110	64	0.157	100	12
17	12	0.007	4	0.031	20	12
18	12	0.090	52	0.082	52	12
A†	Free water	0.193	111	.....	....	....

* This is based on fifteen weekly averages. The averages from some of the soil tanks are for shorter periods, hence are not directly comparable with that for Tank No. 8.

† Tank, 3 by 3 by 3 ft., located 500 ft. northwest of evaporation plots.

To study more in detail the effect on evaporation, of sprinkling the soil surface in quantities that approximate a shower of rain, two soil tanks, Nos. 17 and 18, were so treated. Fig. 3 shows the relation of loss from a free water surface and the losses from Tanks Nos. 17 and 18 during the sprinkled and unsprinkled periods. The quantities of water applied during the sprinkled period are shown in Table 4. For the unsprinkled period, the ratio of loss from the tanks when compared with that from the free water was 11% for Tank No. 17 and 32% for Tank No. 18. The corresponding ratios for the sprinkled period were 33% and 76%, respectively. To what extent the sprinkled water was held in the voids of the sands is not known, but because of the similarity of the rate-of-loss curves representing the soils to that representing the free water, the writer considers it fair to assume that the rate of evaporation loss for the sprinkled period was materially increased. The data given in Table 5 for Tank No. 17 indicate that the percentage of moisture held within the soil mass above the water-table was at or near the maximum capillary capacity, because the quantity used from the storage reservoir of the Mariotte regulation was very small, or showed an increase or an accumulation from the soil tank. Tank No. 18 drew liberally from the supply reservoir to make up for the increased loss due to evaporation.

* "The Principles of Soil Management," by Messrs. T. L. Lyon and E. O. Fippin, p. 198, 1911.

Other experimenters studying evaporation losses from various types of soils have found results varying between rather wide limits. The problem involves too many factors to make possible any correlation which would show other than very general relations.

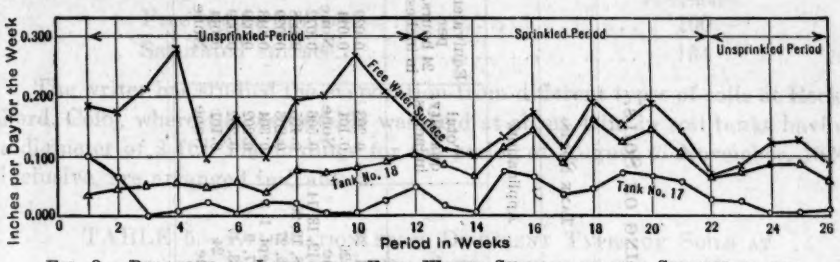

FIG. 3.—RELATION OF LOSS FROM FREE WATER SURFACE TO THE SPRINKLED AND UNSPRINKLED LOSSES FROM TANKS NOS. 17 AND 18.

In studying the evaporation losses from Yolo fine sandy loam, in tanks having diameters of 2 ft. 1 in., with the water-table at the surface and also at 1 in. below the soil surface, Samuel Fortier, M. Am. Soc. C. E., and Mr. S. Beckett, at Davis, Calif., found the following ratio of losses over a period of nine weeks, from August 2 to October 3:*

	Percentage.
Free water surface	100
Saturated surface	84
Water-table at 1 in.	82

The late R. B. Sleight, Assoc. M. Am. Soc. C. E., working at Denver, Colo., with a river-bed sand, in tanks having a nominal diameter of 2 ft., found the evaporation losses from August 4 to October 16 to vary with depth of water-table, as follows:*

	Percentage.
Free water surface	100
Water-table at sand surface	77
Water-table at 3 in.	69
Water-table at 6 in.	64.5
Water-table at 10½ in.	57.7
Water-table at 24 in.	11.3

Mr. Sleight also found, for similar conditions and for a fine sandy loam soil, the following variation of evaporation losses according to depth to water-tables:*

	Percentage.
Free water surface	100
Water-table at 4 in.	88.2
Water-table at 16 in.	79.8
Water-table at 28 in.	62.4
Water-table at 38 in.	33.0
Water-table at 43 in.	7.63
Water-table at 50½ in.	7.24

* These data were taken from "Use of Water in Irrigation," by Samuel Fortier, M. Am. Soc. C. E., New York, 1926.

TABLE 4.—RECORD OF WATER CONSUMPTION FOLLOWING SPRINKLING ON SOILS.

PERIOD.		TANK NO. 17.					TANK NO. 18.				
			Application.					Application.			
Number.	Week.	Loss per day for the week (scale reading),* in inches.	Date.	Total quantity, in inches.	Equivalent per 24 hours, in inches.	Total water consumed per day for the week, in inches.	Loss per day for the week (scale reading), in inches.	Date.	Total quantity, in inches.	Equivalent per 24 hours, in inches.	Total water consumed per day for the week, in inches.
12	July 17–24	0.019	July 21, 23	0.203	0.029	0.048	0.076	July 21, 23	0.203	0.029	0.105
13	July 24–31	+0.001	July 26	0.010	0.014	0.013	0.068	July 26	0.101	0.014	0.082
14	July 31–Aug. 7	0.004	None	None	None	0.004	0.066	None	None	None	0.066
15	Aug. 7–14	0.001	Aug. 10, 11, 12, 13, 14	0.506	0.072	0.073	0.046	Aug. 10, 11, 12, 13, 14	0.506	0.072	0.118
16	Aug. 14–21	0.011	Aug. 15, 18, 19, 20	0.405	0.058	0.069	0.088	Aug. 15, 18, 19, 20	0.405	0.058	0.146
17	Aug. 21–28	+0.004	Aug. 26, 27, 28	0.304	0.043	0.039	0.060	Aug. 26, 27	0.203	0.029	0.089
18	Aug. 28–Sept. 4	0.000	Aug. 30, Sept. 1, 3	0.304	0.043	0.043	0.118	Aug. 30, Sept. 1, 3	0.304	0.043	0.161
19	Sept. 4–11	+0.001	Sept. 6, 7, 8, 11	0.506	0.072	0.071	0.099	Sept. 7, 9	0.202	0.029	0.128
20	Sept. 11–18	+0.003	Sept. 12, 13, 14, 16, 17	0.506	0.072	0.069	0.120	Sept. 13, 17	0.202	0.029	0.149
21	Sept. 18–25	+0.001	Sept. 19, 20, 22, 24	0.405	0.058	0.057	0.092	Sept. 24	0.101	0.014	0.106
22	Sept. 25–Oct. 2	0.014	Sept. 29	0.101	0.014	0.028	0.061	None	None	None	0.061

*Plus sign indicates gain.

M. R. Lewis, M. Am. Soc. C. E., working at Twin Falls, Idaho, with a medium clay loam, in tanks having a diameter of 3 ft., found the following relation from May 27 to October 14:*

	Percentage.
Free water surface	100
Saturated surface	134

The writer has studied the evaporation from different types of soils at Rocky Ford, Colo., where the water-table was held at about 4 in. in soil tanks having a diameter of 2 ft. The findings for the period of August to November, 1924, inclusive, are arranged in Table 5.

TABLE 5.—EVAPORATION FROM DIFFERENT TYPES OF SOILS AT ROCKY FORD, COLORADO.

Location.	Material.	Depth of water-table below surface, in inches.	Ratio, percentage.
	Free water surface	.....	100
Patterson Hollow	Heavy adobe alkaline soil	4.3	19
Experimental plots	Fine sandy loam	5.6	30
Arkansas River	Fine sand mixed with silt, Rocky Ford	4.1	80
Arkansas River	Fine sand mixed with silt, Avondale, Colo	3.8	77
Arkansas River	Fine sand mixed with clay and silt, Las Animas, Colo	4.1	58
Arkansas River	Fine sand mixed with silt, Lamar, Colo	3.8	68

The river sands in this group were substantially of the same general type. The sands in the Rocky Ford experiment were used as test soils in Tanks Nos. 1 to 4, as previously described (Table 1). The results obtained from June to October, 1926, for these same materials are as listed in Table 6.

TABLE 6.—EVAPORATION FROM DIFFERENT TYPES OF RIVER SANDS.

Material.	Depth of water-table below surface, in inches.	Ratio, percentage.
Free water surface	1	100
Rocky Ford, Colo., fine river sand	1	100
Avondale, Colo., fine river sand	1	100
Las Animas, Colo., fine river sand	1	96
Lamar, Colo., fine river sand	1	102

Fig. 4 (*a*) and (*b*) show graphically the trend of the evaporation loss from different soils with increasing depth of water-table. These losses are expressed as a ratio of that from a free water surface. In Fig. 4 (*a*) the four river samples of fine sand with the water-table at 1 in., show a loss equal to that from a free water surface. The loam and heavy adobe soils show a loss of 83 and 75%, respectively, for a depth of 1 in. to the water-table, as compared with that from free water. When the water-table is 6 in. below the

* These data were taken from "Use of Water in Irrigation", by Samuel Fortier, M. Am. Soc. C. E., New York, 1926.

soil surface, the Rocky Ford sand and the adobe soil show a reduction of loss, while the loam soil and both the medium and coarse river sands show a loss equalling that from the free water surface. These three soils throughout the period, June to October, showed a consistently high rate of evaporation. The reason for this persistent high rate of loss is not apparent.

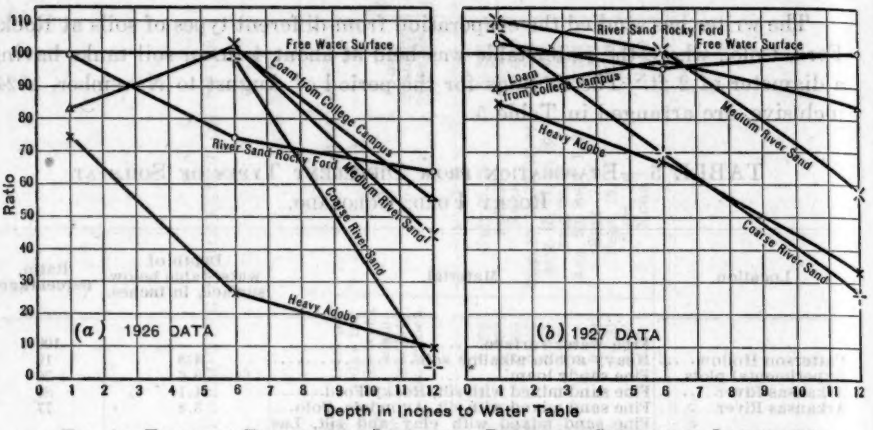

FIG. 4.—TREND OF EVAPORATION LOSS FROM DIFFERENT SOILS WITH INCREASING DEPTH OF WATER-TABLE.

CONCLUSIONS

It is believed that the method of using the Mariotte principle in obtaining the evaporation loss from moist soils, even though there are some apparent inconsistencies in its operation, gives more dependable results than the method of weighing. The results in Table 6 show very marked consistency for fine river sands, which are essentially identical, with the exception of the silt content. Those cited are in evidence of the dependability of the apparatus. The feature of automatic control is highly desirable.

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PROBLEMS CONCERNING ELASTIC STABILITY IN STRUCTURES*

By S. Timoshenko,† Esq.

Synopsis

Sometimes the strength of a structure does not depend on the ultimate strength of the material used, but on the elastic stability of the members. Such problems become of great practical importance in steel structures where materials of high strength are used. Problems of this kind, involving buckling of built-up columns, buckling of a compressed top chord of a half-through span, and stability of webs in compressed members and girders, are considered.

I.—Introduction

The simplest case in which the strength of a structure does not depend on the ultimate strength of the material, but on the elastic stability of the members of the structure, is a strut or a column under the action of a compressive force. If the length of the strut is large in comparison with its cross-sectional dimensions, failure will occur as a result not of a high compressive stress but of elastic instability. The strut may buckle sidewise at a stress less than the yield point of the material.

In steel structures, materials of high strength are used and, as a consequence, the transverse dimensions of members are likely to become small in comparison with the length. With the modern tendency to increase the yield point of structural steels the field of application of theoretical solutions, based on the assumption of a perfectly elastic material, increases and these solutions are, therefore, of practical interest. Several solutions of this kind will be given and their application to such problems as stability of built-up columns, stability of I-beams, and stability of webs of girders, will be considered.

Note.—Written discussion on this paper will be closed in August, 1929.

* Presented at the meeting of the Structural Division, Columbus, Ohio, October 13, 1927.

† Ann Arbor, Mich.

II.—Stability of Built-Up Columns

A solid column with hinged ends will be taken as a fundamental case and the critical loads, P_{cr}, for different built-up columns will be compared with the known critical load for this case, which is:

$$P_{cr} = \frac{E\,I\,\pi^2}{l^2} \qquad (1)$$

in which, $E\,I$ is the flexural rigidity of the column in the plane of buckling, and l is the length of the column.

Critical loads for built-up columns are always less than those for solid columns having the same value of $\frac{E\,I}{l^2}$ and depend on such details as lattice-bars and spacing plates, or battens. The value of critical loads for such columns can always be represented by the equation:

$$P_{cr} = \alpha \frac{E\,I\,\pi^2}{l^2} \qquad (2)$$

in which, α is a factor less than unity.

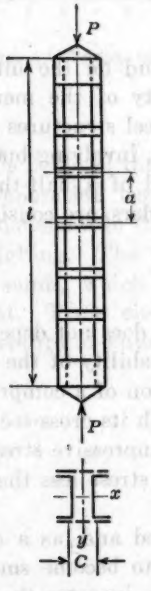

Fig. 1.

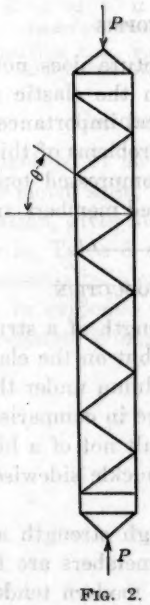

Fig. 2.

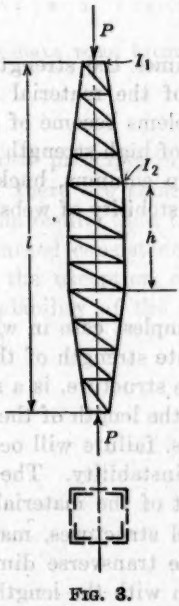

Fig. 3.

Take, for instance, the case shown in Fig. 1. The factor, α, in Equation (2) for this case can be represented in the following form:*

$$\alpha = \frac{1}{1 + \frac{E\,I\,\pi^2}{l^2}\left(\frac{a\,c}{12\,E\,I_2} + \frac{a^2}{24\,E\,I_1}\right)} \qquad (3)$$

* For the derivation of this equation see "Applied Elasticity," by S. Timoshenko and J. M. Lessells, pp. 180–182.

in which,

a = distance between the axes of battens.
c = distance between centers of gravity of channels.
I = moment of inertia of the cross-section about the y-axis.
I_1 = moment of inertia of the cross-section of a channel about the axis through its center of gravity and parallel to the y-axis.
I_2 = moment of inertia of the cross-section of a batten about the axis through its center of gravity and perpendicular to the axis of the column.

It is seen that the strength of the column against buckling depends not only on the value of $\frac{E I}{l^2}$ but also on the rigidity of the channels and battens of which it is made.

Numerical Example.—For a cross-section consisting of the 15-lb., 10-in. channels, let it be assumed that $l = 30$ ft., $a = 36$ in., $c = 7.3$ in., the thickness of battens $= \frac{3}{8}$ in., and the width of battens $= 9$ in. Then, for the cross-section, calculated as a solid column, $I = 123$ in.^{4}, while $I_1 = 2.3$ in.^{4}, and $I_2 = 45.6$ in.^{4}. Substituting in Equation (3), $\alpha = 0.82$. Hence, in this case, the ultimate strength of the built-up column is 0.82 of the critical load of the corresponding solid column.*

If the proportions of a built-up column are such that the compressive stress calculated from Equation (2) is beyond the yield point of the material, the factor, α, will be greater than that given by Equation (3). Hence, the difference in strength between the two types of columns will diminish with a decrease of the ratio, $\frac{l}{r}$. This conclusion can also be deduced from Equation (3). In order to obtain P_{cr} analytically when the stress as calculated by Equation (2) is beyond the yield point of the material, the constant quantities, $E I$ and $E I_1$, in Equation (3), because of the decrease in modulus of elasticity above the yield point, must be replaced by variable quantities diminishing with the increase in compressive stress. At the same time, the rigidity, $E I_2$, of the battens, which are not subjected to compression, remains equal to unity. This means that the value of P_{cr}, calculated from Equation (3), for the cases of short columns, will always be on the safe side.

In the case of a latticed column (Fig. 2), the values of the factor, α, in Equation (2) can be calculated from the equation:

$$\alpha = \frac{1}{\frac{E I \pi^2}{l^2} \frac{1}{E \sin \theta \cos^2 \theta A}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which, A is the cross-sectional area of a lattice-bar.

The latticing is usually so proportioned that the factor, α, differs little from unity. Then, the formula for solid columns can be used.

Consider now a square column shown in Fig. 3. The middle part (of the length, h) has a constant moment of inertia, I, decreasing toward the ends,

* A central application of the compressive force, P, is always assumed.

where it is I_1. Assuming that the latticing is proportioned in such a manner that the column can be considered as solid and of variable cross-section, the critical value of compressive force will be found from the equation:

$$P_{cr} = \beta \frac{E I}{l^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which, β is a numerical factor, the value of which depends on the ratios, $\frac{h}{l}$ and $\frac{I_1}{I}$. Evidently, the value of β must approach π^2 when the ratios, $\frac{h}{l}$ or $\frac{I_1}{I}$, approach unity because then the column becomes almost uniform in cross-section. Some values of the factor, β, are given in Table 1.*

TABLE 1.—VALUES OF THE FACTOR, β, IN EQUATION (5).

Values of $\frac{I_1}{I}$	VALUES OF $\frac{h}{l}$.					
	0.0.	0.2.	0.4.	0.6.	0.8.	1.0.
0.0001	1.000	2.972	4.754	7.658	9.622	π^2
0.2	6.374	7.488	8.611	9.443	9.814	π^2
0.4	7.614	8.420	9.149	9.634	9.888	π^2
0.6	8.512	9.038	9.477	9.736	9.853	π^2
0.8	9.243	9.499	9.699	9.817	9.859	π^2
1.0	π^2	π^2	π^2	π^2	π^2	π^2

III.—BUCKLING OF STRUTS HAVING LATERAL ELASTIC SUPPORTS

This type of problem is encountered, for instance, in considering sidewise buckling of the compressed top chord of a half-through truss span. As a simple case, assume that a strut of uniform cross-section, when buckled under the action of compressive forces, P (Fig. 4), encounters a resistance produced by an elastic medium such that the lateral reactions continuously distributed along the strut are proportional to the corresponding deflections. Because of these lateral reactions the critical load will be greater than that given by Equation (1). The magnitude of this load can be found from a consideration of the potential energy of the system.† The critical value of the compressive force, P, is that at which the work done during sidewise buckling by the compressive forces is equal to the potential energy of bending of the strut, together with the energy of deformation of the elastic medium. For the potential energy of bending of the strut the following equation applies:

$$V_1 = \frac{E I}{2} \int_0^l \left(\frac{d^2 y}{d x^2}\right)^2 d x \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

In considering the potential energy of the elastic medium a factor, K, called the modulus of the medium is taken such that the product, $K y$, represents the reaction of the medium per unit length of the strut when the deflection is equal to y. Then the work done by this reaction during deflection

* This table was calculated by Prof. A. Dinnik, Polytechnical Inst., Ekaterinoslaw (Russia).

† For discussion of this energy method, see "Applied Elasticity," p 169.

will be $-\frac{1}{2} K y^2$ and, consequently, the potential energy of the deformed elastic medium will be:

$$V_2 = \frac{K}{2} \int_0^l y^2 \, dx \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

FIG. 4.

FIG. 5.

The work done by the compressive forces during buckling will be obtained by multiplying P by the displacement of the end, A, with respect to B, which occurs during sidewise buckling of the strut. Taking into consideration the fact that the difference between the element, ds, of the deflection curve and its vertical projection, dx $\left(\text{since } \frac{dy}{dx} \text{ is small}\right)$, is equal to:

$$ds - dx = dx \sqrt{1 + \left(\frac{dy}{dx}\right)^2} - 1 = \frac{1}{2} \left(\frac{dy}{dx}\right)^2 dx$$

the work, T, produced by the compressive forces during buckling will be:

$$T = \frac{P}{2} \int_0^l \left(\frac{dy}{dx}\right)^2 dx \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Now from Equations (6), (7), and (8), the condition for the critical load will be:

$$\frac{1}{2} P \int_0^l \left(\frac{dy}{dx}\right)^2 = \frac{1}{2} EI \int_0^l \left(\frac{d^2y}{dx^2}\right)^2 dx + \frac{1}{2} K \int_0^l y^2 \, dx \ldots\ldots (9)$$

If the elastic medium has a small rigidity so that the factor, K, is small, the deflection curve will be the same as in the case of a strut without lateral support. Therefore, it can be represented by:

$$y = c \sin \frac{\pi x}{l} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Substituting this in Equation (9),

$$P_{cr} = \frac{EI\pi^2}{l^2} \left(1 + \frac{Kl^4}{EI\pi^4}\right) \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

The second term in the brackets represents the effect of the elastic medium on the critical load.

In the case of a medium of greater rigidity, the deflection curve of the buckled strut has a more complicated shape consisting of several waves (Fig. 5). With an increase in rigidity of the medium, the number of waves of the deflection curve increases, but each wave is a simple sine wave and for calculation of critical loads Equation (11) can be used. To do this it is only necessary to substitute in this equation, instead of the length, l, $\frac{l}{m}$, in which, m denotes the number of waves. Then:

$$P_{cr} = \frac{E\,I\,\pi^2}{l^2}\left(m^2 + \frac{K\,l^4}{E\,I\,\pi^4\,m^2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

In each case, m must be chosen in such a manner as to make the sum in the bracket a minimum. This number, which depends on the rigidity of the medium, increases with the increase of the quantity, K, and can be determined in the following manner: Let K be the rigidity at which the number of waves in the deflection curve of the strut changes from m to $m + 1$. Then, the sum in the brackets of Equation (12) will remain unchanged by replacing m by $(m + 1)$, and, consequently,

$$m^2 + \frac{K\,l^4}{E\,I\,\pi^4\,m^2} = (m+1)^2 + \frac{K\,l^4}{E\,I\,\pi^4\,(m+1)^2} \quad \ldots\ldots\ldots\ldots (13)$$

from which,

$$m^2\,(m+1)^2 = \frac{K\,l^4}{E\,I\,\pi^4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

In this manner for each value of m the limiting value for the rigidity, K, can be calculated. For a value of K slightly less than this calculated value, the deflection curve will have m waves, while for a value of K slightly greater the deflection curve will have $m + 1$ waves.

If the rigidity, K, or the length, l, are very large, the number, m, will also be large and, neglecting unity in comparison with m in Equation (14),

$$m^4 = \frac{K\,l^4}{E\,I\,\pi^4}$$

Substituting this in Equation (12),

$$P_{cr} = \frac{2\,E\,I\,\pi^2\,m^2}{l^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

The critical load in this case is thus twice as great as in the case of a strut without lateral support and having a length equal to,

$$\frac{l}{m} = \sqrt[4]{\frac{E\,I\,\pi^4}{K}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

IV.—Buckling of Compressed Top Chord of a Half-Through Truss Span

To determine the critical compressive force for the top chord of a half-through truss span is much more complicated because the compressive forces are distributed along the chord, the section of the chord is not constant, and instead of a continuous elastic medium there are lateral elastic supports represented by vertical truss members. Consider the simplest case of a through-

truss span of a constant height, h (Fig. 6), and assume that the top chord has a uniform cross-section and that all the vertical truss members are of the same section except the end verticals which are so rigid that the ends of the top chord do not move laterally during buckling of the chord.

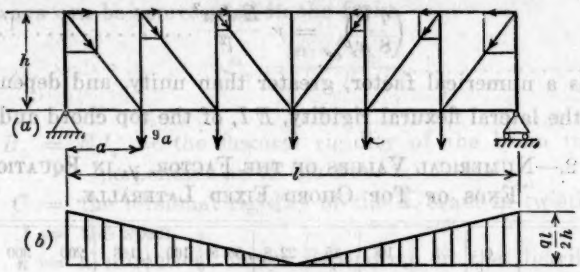

FIG. 6.

Assuming that the truss is subjected to the action of a uniformly distributed load of intensity, q, the tensile forces in the diagonals and their horizontal components, producing compression in the top chord, will be proportional to the distances from the middle of the span. Replacing the concentrated compressive forces acting on the top chord by equivalent continuously distributed forces, the distribution of the compressive forces will be as shown by the shaded area in Fig. 6 (b). The maximum compression will be at the middle of the chord and will be equal to $\frac{q\,l^2}{8\,h}$. The critical value of this compressive force corresponding to the beginning of sidewise buckling of the top chord can be found in the same manner as in the simple case of a strut in an elastic medium, already discussed. It is only necessary to replace the concentrated elastic reactions of the verticals, opposing the lateral deflection of the compressed chord, by continuously distributed reactions of an equivalent elastic medium. The modulus, K, of the equivalent medium will be:

$$K = \frac{Q}{a} \qquad (17)$$

in which, Q is the lateral force which, if applied to the vertical member at the top chord panel point (neglecting for the moment the lateral resistance of the chord) will produce a lateral deflection of the vertical member equal to unity; and a is the distance between the verticals.

Assuming, for instance, that the lower ends of the verticals are absolutely fixed and putting $E\,I_1$ equal to the flexural rigidity of the verticals in a lateral direction, the force, Q, will be obtained from the expression,*

$$\frac{Q\,h^3}{3\,E\,I_1} = 1$$

from which,

$$Q = \frac{3\,E\,I_1}{h^3} \qquad (18)$$

* The bending of the cross-beam of the floor structure and the effect of the compressive force in the verticals on the deflection are neglected in this equation.

Calculating K from Equations (17) and (18), and using an energy equation similar to Equation (9), it will be found (see Appendix) that the critical value of the maximum compressive force in the top chord can be represented by:

$$\left(\frac{q\,l^2}{8\,h}\right)_{cr} = \gamma\,\frac{E\,I\,\pi^2}{l^2} \qquad (19)$$

in which, γ is a numerical factor, greater than unity, and depending on the modulus, K, the lateral flexural rigidity, $E\,I$, of the top chord and the span, l.

TABLE 2.—NUMERICAL VALUES OF THE FACTOR, γ, IN EQUATION (19), ENDS OF TOP CHORD FIXED LATERALLY.

$\frac{K\,l^4}{16\,E\,I}$ =	0	5	10	15	22.8	56.5	100	163	200	300	500	1 000
γ =	2.06	3.63	5.10	6.37	7.58	9.51	11.9	14.9	16.5	19.8	24.0	33.0

By using Table 2 the factor, γ, can be found for a given value of $\frac{K\,l^4}{16\,E\,I}$. Then, the critical compressive force in the chord will be the same as for a simple strut, the length of which is equal to $\frac{l}{\sqrt{\gamma}}$. For $K = 0$ the case is that of a strut without lateral supports and loaded as shown in Fig. 6 (*b*). The critical compressive force is then about twice as great ($\gamma = 2.07$) as for a strut compressed by forces applied at the ends.

This method can be applied also in the case when end verticals are of the same rigidity as the intermediate verticals and the ends of the top chord are free to move laterally during buckling. Equation (19) can be used also in this case, but the numerical values of the factor, γ, are, however, somewhat different (Table 3).

TABLE 3.—NUMERICAL VALUES OF THE FACTOR, γ, IN EQUATION (19), ENDS OF TOP CHORD FREE TO MOVE LATERALLY.

$\frac{K\,l^4}{16\,E\,I}$ =	0	1	3	5	10	15	20	50	100	150	200	300	500	1 000
γ =	0	0.203	0.610	1.015	2.02	3.03	3.40	5.38	8.64	13.05	14.70	17.98	23.6	32.3

With an increase in the quantity, $\frac{K\,l^4}{16\,E\,I}$, the values of γ approach those given in Table 2 for rigid end verticals. This result is easily understood since for the compressed top chord the deflection curve during lateral buckling may have several waves as in the case of a strut in an elastic medium (Fig. 5). The number of waves increases with the rigidity, K; hence when this number is large, the critical load is no longer affected substantially by a variation in the end conditions.

V.—Lateral Buckling of I-Beams

It is known that, in the absence of lateral supports, an I-beam bent in the plane of the web may prove to be insufficiently stable. If the loads are increased beyond a certain critical limit, it buckles sidewise and collapses. The critical load can be found by the energy method; for a beam on two supports it always can be represented in the form:

$$P_{cr} = \frac{n\sqrt{B_2 C}}{l^2} \quad \ldots\ldots(20)$$

in which,

$B_2 = E I_2$ = the flexural rigidity of the beam in the direction perpendicular to the web.

C = the torsional rigidity of the I-beam in twisting.

l = the span.

n = a numerical factor depending on the distribution of load, on the manner of fastening the ends of the beam, and on the magnitude of the ratio, $\frac{C l^2}{B_2 h^2}$, h being the depth of the beam.

Tables of the factor, n, for various cases, and the application of Equation (20) have been discussed previously.* In Fig. 7,† beams of very narrow

FIG. 7.

rectangular cross-section, instead of I-section, are taken. For such a beam supported at the ends, Fig. 7 (*a*), the critical load, P_{cr}, at which sidewise buckling begins is:

$$P_{cr} = \frac{16.9\sqrt{B_2 C}}{l^2} \quad \ldots\ldots(21)$$

In the case of a cantilever beam (Fig. 7 (*b*)):

$$P_{cr} = \frac{4.01\sqrt{B_2 C}}{l^2} \quad \ldots\ldots(22)$$

If h is the depth, t, the thickness of the web, and G, the modulus of shear,

$$B_2 = \frac{E h t^3}{12} \text{ and } C = \frac{1}{3} G h t^3$$

Substituting these in Equations (21) and (22), the critical load for each particular case easily can be calculated.

VI.—Buckling of Compressed Plates

A.—In compression members of large cross-section, buckling of compressed plates may be of practical importance. Consider, for instance, the cross-

* *Transactions*, Am. Soc. C. E., Vol. 87 (1924), p. 1247.

† These cases have been discussed by L. Prandtl, in his dissertation, "Kipperscheinungen," München, 1899.

sections given in Fig. 10. By diminishing the thickness of the column wall the member will fail under compression by the sidewise buckling of the plates themselves and not by the deflection of the column as a whole. The sides of columns may be considered as long rectangular plates under compression and with various conditions of fixity along the edges. In the simplest case a rectangular plate simply supported is compressed by forces uniformly distributed along the edges, $x = 0$ and $x = a$ (Fig. 8). When the compressive stress exceeds certain limits (critical compressive stress), the flat form of the plate becomes unstable and lateral buckling will occur. By application of the energy method, the equation for the critical stress will be:

$$p_{cr} = \alpha_1 p_e \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

in which,

$$p_e = \frac{E \pi^2 t^2}{12 h^2 (1 - \sigma^2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

h = the width of the plate.

σ = Poisson's ratio for the material.

α_1 = a numerical factor depending on the ratio, $\frac{a}{b}$.

It is easy to see that p_e represents the critical compressive stress as obtained by an equation analogous to Equation (1) for a strip having the length, h, and the thickness, t, equal to the thickness of the plate. In Table 4 several values of α_1 for various ratios, $\frac{a}{h}$, are given.

TABLE 4.—VALUES OF α_1 IN EQUATION (23) FOR RECTANGULAR PLATE SIMPLY SUPPORTED ON ALL SIDES.

$\frac{a}{h}$ =	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0
α_1 =	8.41	5.14	4.20	4.00	4.13	4.47	4.20	4.04	4.00
p_{cr}, in pounds per square inch =	22 800	14 000	11 400	10 900	11 200	12 100	11 400	11 000	10 900

It is seen that α_1 is a minimum for a square plate. If a long plate buckles under compression it subdivides in squares or in rectangles, almost square. For each of these rectangles Equation (23) can be applied, the length, a, being the distance between the two consecutive nodal lines parallel to the y-axis. For a longer plate, say, when $\frac{a}{b} > 3$, a value, $\alpha_1 = 4$, always represents a good approximation.

In Table 4 the values, p_{cr}, are calculated on the assumption that $E = 30 \times 10^6$ lb. per sq. in., $\sigma = 0.3$, and $\frac{h}{t} = 100$. Taking into consideration that the critical stress is proportional to $\frac{t^2}{h^2}$ (Equation (24)), the

critical stress for any value of $\frac{h}{t}$, different from 100, is obtained by multiplying the tabular values by $10^4 \frac{t^2}{h^2}$. Take, for instance, a long steel plate having a yield point of 40 000 lb. per sq. in. To determine the value of the ratio, $\frac{h}{t}$, at which the critical stress becomes equal to the yield point, assuming $\alpha_1 = 4$: From Table 4,

$$10\,900 \times 10^4 \times \frac{t^2}{h^2} = 40\,000$$

that is,

$$\frac{h}{t} = \sqrt{\frac{10\,900}{4}} = 51 \pm$$

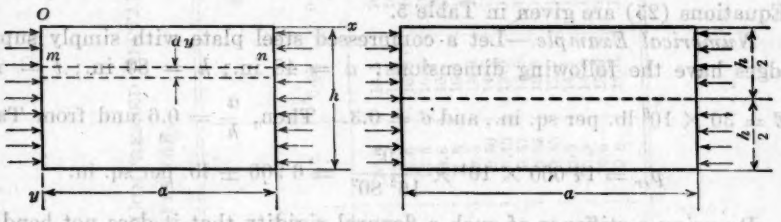

FIG. 8.

FIG. 9.

For larger values of the ratio, $\frac{h}{t}$, failure will occur as a result of instability and at a compressive stress less than the yield point. This means that in consequence of lateral buckling the complete strength of the material of the plate will not be utilized.

B.—By properly increasing the thickness of the plate, the buckling will not occur below the yield point; but such a uniform increase in thickness is not always the most economical solution. Sometimes for compressed plates of considerable width, it appears more advantageous to keep the thickness as small as possible and to increase the stability against buckling by introducing longitudinal stiffeners. Such methods of reinforcement are used sometimes in shipbuilding, where the economy in the proper weight of the hull is of great practical importance. In proportioning such reinforcement, it is important to know how the stability of the reinforced plate depends on the flexural rigidity and cross-sectional area of the stiffeners. This problem can be solved by using the energy method.*

If the plate is reinforced by one longitudinal stiffener at the middle of the width (Fig. 9), the maximum reinforcement occurs when the flexural rigidity of the stiffener is sufficient to prevent it from bending during buckling of the plate. Then each half of the plate will be in the condition of a plate of the width, $\frac{h}{2}$. At smaller values of the flexural rigidity the stiffener

* For detailed investigation of this question, see the writer's paper in *Der Eisenbau*, Vol. 12 (1921), p. 147.

bends with the plate during buckling and the value of the critical compressive stress will depend on the ratios:

$$\frac{a}{h};\ \frac{B}{h\,D};\ \frac{A}{h\,t} \qquad (25)$$

in which,

$B = E\,I$ = flexural rigidity of the stiffener in bending at right angles to the plane of the plate.

$h\,D = h\,\dfrac{E\,t^3}{12\,(1 - \sigma^2)}$ = the flexural rigidity of the plate.

A = cross-sectional area of the stiffener.

$h\,t$ = cross-sectional area of the plate.

For calculating the critical compressive stress, Equation (23) can be used. The values of the numerical factor, α_1, for various values of the ratios in Equations (25) are given in Table 5.

Numerical Example.—Let a compressed steel plate with simply supported edges have the following dimensions: $a = 48$ in.; $h = 80$ in.; $t = \frac{5}{16}$ in.; $E = 30 \times 10^6$ lb. per sq. in., and $\sigma = 0.3$. Then, $\dfrac{a}{h} = 0.6$ and from Table 4,

$$p_{cr} = 14\,000 \times 10^4 \times \frac{9^2}{16^2\,80^2} = 6\,900 \pm \text{ lb. per sq. in.}$$

By using a stiffener of such a flexural rigidity that it does not bend while the plate is buckling, the critical stress becomes:

$$p_{cr} = 11\,200 \times \frac{9^2}{16^2\,40^2} = 22\,100 \pm \text{ lb. per sq. in.}$$

From Table 5 it is seen that $\left(\text{for } \dfrac{a}{h} = 0.6\right)$ such a condition can be obtained by using a stiffener, for which $\dfrac{B}{h\,D} = 5$ and $\dfrac{A}{h\,t} < 0.2$. In calculating the quantity, B, note that the stiffener is riveted to a plate of great width and this results in a considerable increase of rigidity. By taking, for instance, the stiffener in the form of a channel, or a Z, riveted to the plate by its flange, the center of gravity of the section consisting of the stiffener and the plate will be very near to the surface of the plate, and the moment of inertia of the cross-section of the stiffener with respect to the axis coinciding with the outer surface of the flange must be taken in calculating B. Using two longitudinal stiffeners, sub-dividing the plate into three equal parts, Equation (23) still applies for calculating the critical compressive stress. The corresponding values of the factor, α_1, are given in Table 6.

C.—When the edges of the plate parallel to the direction of compression, instead of being simply supported, are built in, the stability of the plate is increased. For calculating the critical values of compressive stress, Equation (23) can be used, but the values of the factor, α_1, must be taken from Table 7. It is seen that the minimum value of the factor, α_1, corresponds to the ratio, $\dfrac{a}{h} = 0.7 \pm$, so that a long compressed plate, when it buckles, subdivides

TABLE 5.—VALUES OF THE FACTOR, α_1, EQUATION (23) FOR A COMPRESSED RECTANGULAR PLATE REINFORCED BY A LONGITUDINAL STIFFENER.

$\frac{a}{h}$	$\frac{A}{ht} = 0.05.$	$\frac{B}{hD} = 5.$ 0.1.	0.2.	10. 0.05.	0.1.	0.2.	15. 0.05.	0.1.	0.2.	20. 0.05.	0.1.	0.2.	25. 0.05.	0.1.	0.2.
0.6	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
0.8	15.4	14.6	13.0	16.8	16.8	16.8	16.8	16.8	16.8	16.8	16.8	16.8	16.8	16.8	16.8
1.0	12.0	11.1	9.72	16	16	15.8	16.0	16	16.0	16.0	16.0	16.0	16.0	16.0	16.0
1.2	9.83	9.06	7.88	15.3	14.2	12.4	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
1.4	8.62	7.91	6.82	12.9	12.0	10.3	16.1	15.7	13.6	16.1	16.1	16.1	16.1	16.1	16.1
1.6	8.01	7.38	6.32	11.4	10.5	9.05	14.7	13.6	11.8	16.1	16.1	14.4	16.1	16.1	16.1
1.8	7.84	7.19	6.16	10.6	9.70	8.35	13.2	12.2	10.5	15.9	14.7	12.6	16.2	16.2	14.7
2.0	7.96	7.29	6.24	10.2	9.35	8.03	12.4	11.4	9.80	14.6	13.4	11.6	16.0	15.4	13.3
2.2	8.28	7.58	6.50	10.2	9.30	7.99	12.0	11.0	9.45	13.9	12.7	10.9	15.8	14.5	12.4
2.4	8.79	8.06	6.91	10.4	9.49	8.15	11.9	10.9	9.37	13.5	12.4	10.6	15.1	13.8	11.9
2.6	9.27	8.50	7.28	10.8	9.86	8.48	12.1	11.1	9.53	13.5	12.4	10.6	14.8	13.6	11.6
2.8	8.62	7.91	6.81	11.4	10.4	8.94	12.5	11.5	9.85	13.7	12.6	10.8	14.8	13.6	11.6
3.0	8.31	7.62	6.53	12.0	11.1	9.52	13.1	12.0	10.3	14.1	13.0	11.1	15.2	13.9	11.9
3.2	8.01	7.38	6.32	11.4	10.5	9.05	13.9	12.7	10.9	14.8	13.5	11.6	15.6	14.3	12.3
3.6	7.84	7.19	6.16	10.6	9.70	8.35	13.2	12.2	10.5	15.9	14.7	12.6	16.2	15.7	13.5
4.0	7.96	7.29	6.24	10.2	9.35	8.03	12.4	11.4	9.8	14.6	13.4	11.6	16.0	15.4	13.3

into rectangles approaching these proportions. For a plate of considerable length, say, $\frac{a}{h} > 2$, $\alpha_1 = 7.00$ is always a good approximation.

TABLE 6.—VALUES OF THE FACTOR, α_1, IN EQUATION (23) FOR A RECTANGULAR PLATE REINFORCED BY TWO LONGITUDINAL STIFFENERS.

$\frac{a}{h}$	$\frac{B}{h\,D} = \frac{10}{3}$		5		$\frac{20}{3}$		10	
	$\frac{A}{h\,t} = 0.05$	0.1	0.05	0.1	0.05	0.1	0.05	0.1
0.6	26.8	24.1	36.4	33.2	36.4	36.4	36.4	36.4
0.8	16.9	15.0	23.3	20.7	29.4	26.3	37.2	37.1
1.0	12.1	10.7	16.3	14.5	20.5	18.2	28.7	25.6
1.2	9.61	8.51	12.6	11.2	15.5	13.8	21.4	19.0
1.4	8.32	7.36	10.5	9.32	12.7	11.3	17.2	15.2
1.6	7.70	6.81	9.40	8.31	11.1	9.82	14.5	12.8
1.8	7.51	6.64	8.85	7.83	10.2	9.02	12.9	11.4
2.0	7.61	6.73	8.70	7.69	9.78	8.65	11.9	10.6

D.—When one longitudinal side ($y = 0$) of the plate shown in Fig. 8 is built in and another ($y = h$) is free, the critical value of the uniform compressive stress can be calculated again from Equation (23). The values of the numerical factor, α_1, for this case are given in Table 8.

TABLE 7.—VALUES OF THE FACTOR, α_1, IN EQUATION (23) FOR A RECTANGULAR PLATE, THE TWO SIDES OF WHICH, PARALLEL TO COMPRESSION, ARE BUILT IN AND TWO OTHERS ARE SIMPLY SUPPORTED.

$\frac{a}{h}$ =	0.4	0.5	0.6	0.7	0.8	0.9	1.0
α_1 =	9.44	7.69	7.05	7.00	7.29	7.83	7.69
p_{cr}, in pounds per square inch, for $\frac{h}{t} = 100$ =	25 000	20 800	19 100	19 000	19 800	21 200	20 800

It is seen that the minimum value of the factor, α_1, corresponds to a value of the ratio, $\frac{a}{h}$, about 1.7. A long plate, while buckling, will subdivide in rectangles approaching these proportions. Taking $\alpha_1 = 1.33$ and the yield point as 40 000 lb. per sq. in., for a long steel plate ($E = 30 \times 10^6$ lb. per sq. in.; $\sigma = 0.3$), the limiting value of the ratio, $\frac{h}{t}$, at which the critical stress and yield point coincide, will be:

$$p_{cr} = 1.33 \frac{E\,\pi^2}{12\,(1 - \sigma^2)} \frac{t^2}{h^2} = 40\,000$$

from which, $\frac{h}{t} = 30 \pm$.

TABLE 8.—VALUES OF THE FACTOR, α_1, FOR A COMPRESSED RECTANGULAR PLATE, ONE OF THE LONGITUDINAL EDGES OF WHICH IS BUILT IN AND ANOTHER FREE.

$\frac{a}{h}$ =	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.2	2.4
α_1 =	1.70	1.56	1.47	1.41	1.36	1.34	1.33	1.33	1.34	1.36	1.38	1.45	1.47

For a larger value of the ratio, $\frac{h}{t}$, the buckling of the plate will occur at a stress less than the yield point of the material and for improving the conditions the reinforcement of the free edge of the plate by riveting a stiffener becomes necessary. By using Tables 4, 5, and 8, the necessary thickness of steel sheets for built-up sections of compressed members, such as those shown in Fig. 10, can easily be calculated.

FIG. 10.

VII.—STABILITY OF THE WEB OF A PLATE GIRDER

In the design of plate girders, the thickness of the web is usually calculated for safety in shearing. The calculation does not give sufficient thickness and in order to prevent buckling usually it is necessary to increase the thickness and to use stiffeners, without considering the stability of the stressed web. This results in types of construction having widely varying factors of safety with respect to buckling. For example, plate girders of one large American railroad system, with a span, $l = 100$ ft., and with a depth, $h = 10$ ft., have a web thickness, t, equal to $\frac{5}{8}$ in., while similar girders of another railroad have a web thickness of $\frac{7}{16}$ in. near the supports and only $\frac{3}{8}$ in. at the middle. These girders, both satisfying American specifications, have very different factors of safety with regard to the buckling of the web.

TABLE 9.—VALUES OF THE FACTOR, α_1, IN EQUATION (23) FOR A RECTANGULAR PLATE SUPPORTED AT THE EDGES AND SUBMITTED TO SHEAR.

$\frac{h}{a}$ =	1	1.2	1.4	1.5	1.6	1.8	2.0	2.5	3.0
α_1 =	9.42	8.0	7.3	7.1	7.0	6.8	6.6	6.3	6.1

A more satisfactory design can be obtained from a consideration of the stability of the stressed web. Near the supports the shearing force is the most important factor, and stresses due to bending can be neglected. Considering the part of the web between two stiffeners as a rectangular plate with sup-

ported edges, subjected only to the action of shear (Fig. 11), the critical value of shearing stress can be found from consideration of the energy, and the equation for calculating this value has the same form as Equation (23). The corresponding values of the numerical factor, α_1, are given in the Table 9.

Applying these results to steel plates ($E = 30 \times 10^6$ lb. per sq. in., $\sigma = 0.3$), and assuming the distance between the stiffeners equal to 5 ft., the critical values of shearing stress for different depths, h, and different thicknesses, t, are as given in Table 10.

TABLE 10.—Critical Shearing Stresses, in Pounds per Square Inch.

Values of h, in feet.	Values of t, in Inches.			
	⅜	7/16	½	9/16
5	9 980	13 600	17 700	22 400
7	7 730	10 500	13 700	17 400
10	6 990	9 510	10 400	15 700

Assuming the factor of safety as 2 and the average shearing stress at the support as 5 000 lb. per sq. in., Table 10 shows that for both the plate girders mentioned, a thickness of ½ in. will insure the stability of the web near the supports.*

Fig. 11.

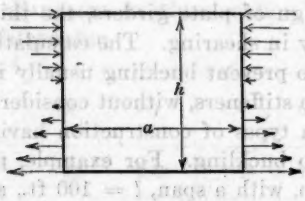

Fig. 12.

It was assumed that the buckling of the web was not accompanied by bending of the stiffeners. This assumption will be true only when the stiffeners have a sufficient flexural rigidity. The necessary magnitude of the moment of inertia of the cross-section of the stiffeners can be found from the potential energy of bending of these stiffeners, giving the data of Table 11.†

The quantity, δ, of Table 11 denotes the ratio, $E\,I$: $\dfrac{E\,a\,t^3}{12\,(1-\sigma^2)}$, in which, $E\,I$ is the necessary flexural rigidity of the stiffeners, and $\dfrac{E\,a\,t^3}{12\,(1-\sigma^2)}$ is the flexural rigidity of the part of the web between two consecutive stiffeners.

* Under actual conditions the factor of safety will be greater than 2, because the edges of the plate are assumed as simply supported and the stiffening effect of fixed edges is neglected.

† See the writer's paper in *Der Eisenbau*, Vol. 12 (1921), p. 147.

TABLE 11.

$\frac{a}{h}$ =	1.0	0.750	0.625	0.500
δ =	3.3	11.6	25.2	60

Taking, for instance, $a = 5$ ft., $h = 10$ ft., and $t = \frac{1}{2}$ in.:

$$I = 60 \frac{\left(\frac{1}{2}\right)^3}{12\ (0.91)} = 41 \pm \text{in.}^4$$

This is the necessary value for the moment of inertia of the cross-section of the stiffeners.

At the middle of the span the shearing stresses can be neglected in comparison with normal stresses. Then, the part of the web between two stiffeners will be in the condition represented in Fig. 12. Applying the same general energy method, Equation (23) will be found for calculating critical values of bending stresses. The corresponding values of the factor, α_1, are given in Table 12.

TABLE 12.—VALUES OF THE FACTOR, α_1, FOR A SIMPLY SUPPORTED RECTANGULAR PLATE STRESSED AS SHOWN IN FIG. 12.

$\frac{a}{h}$ =	0.40	0.50	0.60	0.667	0.75	0.80	1.0	1.5
α_1 =	29.1	25.6	24.1	23.9	24.1	24.4	25.6	24.1

The minimum value of α_1 will be for the ratio, $\frac{a}{h}$, equal to 0.67. Evidently, however, the placing of stiffeners closer together at the middle of the span will not affect materially the stability of the web at this point.

Applying Table 12 in the case of the plate girder having $a = 5$ ft., $h = 10$ ft., and $t = \frac{3}{8}$ in. at the middle,

$$p_{cr} = 25.6 \frac{E\left(\frac{3}{8}\right)^2}{(12)\ (120)^2\ (0.91)} = 6{,}800 \pm \text{lb. per sq. in.}$$

This stress is smaller than that generally considered safe for plate girders. It was assumed that the edges were simply supported; in practice they are always rigidly connected with the flanges. Therefore, the actual critical stresses will be somewhat greater than the theoretical. Nevertheless, probably some buckling of the web occurs under ordinary loading conditions. This buckling does not represent an immediate danger to the girder, but it causes an overstressing of the girder flanges and of the rivets, which is undesirable.

APPENDIX

Stability of the Compressed Top Chord of a Half-Through Truss Span

Considering the compressed top chord of a half-through bridge as a strut of uniform cross-section having hinged ends, and assuming that the compressive forces are continuously distributed, as shown by the shaded area in Fig. 6(*b*), the critical value of the compressive force at the middle can be calculated by using the energy method. In this manner an equation analogous to Equation (9) can be established. Equations (6) and (7) for the potential energy of bending and the potential energy of deformation of the elastic medium can be used also in this case. In calculating the work done by the compressive forces during lateral buckling (Fig. 13) assume that Support B is on rollers so that during buckling it moves toward Support A. Due to displacement of one element, dx, of the strut (Fig. 13) the loads acting on the right-hand portion of the strut are displaced toward Support A by an amount equal to $ds - dx = \frac{1}{2}\left(\frac{dy}{dx}\right)^2 dx$, and the work produced during this displacement will be,

$$-\frac{1}{2}\left(\frac{dy}{dx}\right)^2 dx \int_x^l \frac{q}{2h}\frac{l}{}\left(1-\frac{2x}{l}\right) dx = \frac{q}{4h}\left(\frac{dy}{dx}\right)^2 (l-x)\,x\,dx$$

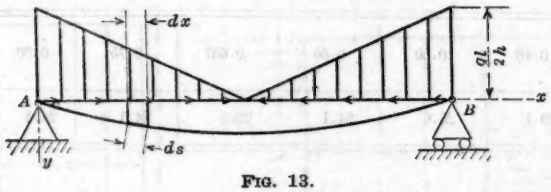

Fig. 13.

The complete work given by the compressive forces during bending will be:

$$T = \frac{q}{4h}\int_0^l \left(\frac{dy}{dx}\right)^2 (l-x)\,x\,dx \quad \ldots\ldots\ldots\ldots\ldots\ldots (A)$$

so that the equation for determining the critical value of the compressive force, $\frac{q}{8}\frac{l^2}{h}$, will have the form:

$$\frac{q}{4h}\int_0^l \left(\frac{dy}{dx}\right)^2 (l-x)\,x\,dx = \frac{1}{2}EI\int_0^l \left(\frac{d^2y}{dx^2}\right)^2 dx + \frac{1}{2}K\int_0^l y^2\,dx \quad \ldots\ldots\ldots\ldots\ldots\ldots (B)$$

The deflection curve in this case is not a simple sine curve, but it can always be assumed to be represented in the form of the series:

$$y = a_1 \sin\frac{\pi x}{l} + a_2 \sin\frac{2\pi x}{l} + a_3 \sin\frac{3\pi x}{l} + \ldots\ldots\ldots\ldots (C)$$

each term of which satisfies the conditions at the ends of the strut. Substituting this series in Equation (B) the right-hand member becomes:

$$\frac{1}{2} E I \int_0^l \left(\frac{d^2 y}{d x^2}\right)^2 d x + \frac{1}{2} K \int_0^l y^2 d x$$

$$= \frac{E I \pi^4}{4 l^3} \sum_{n=1,2,3\ldots.} n^4 a_n^2 + \frac{K l}{4} \sum_{n=1,2,3\ldots.} a_n^2 \quad \ldots\ldots\ldots\ldots (D)$$

In calculating the left-hand member of Equation (B), the following formulas will be used:

$$\int_0^l x \cos^2 \frac{m \pi x}{l} d x = \frac{l^2}{4}$$

$$\int_0^l x^2 \cos^2 \frac{m \pi x}{l} d x = \frac{l^3}{6} + \frac{l^3}{4 \pi^2 m^2}$$

in which, m is an integer.

$$\int_0^l x^2 \cos \frac{n \pi x}{l} \cos \frac{m \pi x}{l} d x = \frac{2 l^3}{\pi^2} \frac{(m^2 + n^2)}{(m^2 - n^2)} 2 (-1)^{m+n}$$

in which, m and n are both integers.

$$\int_0^l x \cos \frac{n \pi x}{l} \cos \frac{m \pi x}{l} d x = 0$$

when $m + n$ is an even number; and,

$$\int_0^l x \cos \frac{n \pi x}{l} \cos \frac{m \pi x}{l} d x = -\frac{2 l^2}{\pi^2} \frac{m^2 + n^2}{(m^2 - n^2)^2}$$

when $m + n$ is an odd number.

Then,

$$\frac{q}{4 h} \int_0^l \left(\frac{d y}{d x}\right)^2 (l - x) x d x = \frac{q l}{4 h} \left\{ \sum_{n=1,2,3\ldots.} a_n^2 \left(\frac{n^2 \pi^2}{12} - \frac{1}{4}\right) \right.$$

$$\left. - 4 \sum_{(m+n)\text{ even}\ldots.} \sum a_n a_m \frac{n m (m^2 + n^2)}{(m^2 - n^2)^2} \right\} \quad \ldots\ldots\ldots\ldots\ldots (E)$$

Substituting values from Equations (D) and (E) in Equation (B),

$$\frac{q l^2}{8 h} = \frac{\frac{E I \pi^4}{8 l^2} \sum_{n=1,2,3\ldots.} n^4 a_n^2 + \frac{K l^2}{8} \sum_{n=1,2,3\ldots.} a_n^2}{\sum_{n=1,2,3\ldots.} a_n^2 \left(\frac{n^2 \pi^2}{12} - \frac{1}{n}\right) - 4 \sum_{(m+n)\text{ even}\ldots.} \sum a_n a_m \frac{n m (m^2 + n^2)}{(m^2 - n^2)^2}} \quad \ldots (F)$$

In order to get the critical value of the compressive force, there only remains the coefficients, $a_1, a_2, a_3 \ldots$, to be determined in such a manner that Equation (F) will be a minimum. By making the derivatives of this expression with respect to $a_1, a_2, a_3 \ldots$, equal to zero, a system of equations will be found having the following form:

$$a_n \left[\pi^2 n^4 + \alpha \pi^2 - \gamma \left(\frac{2}{3} \pi^2 n^2 - 2\right)\right]$$

$$+ 16 \gamma \sum_{(m+n)\text{ even}} \sum a_m \frac{n m (m^2 + n^2)}{(m^2 - n^2)^2} = 0 \quad \ldots\ldots\ldots\ldots (G)$$

in which, for simplification, the following notations are used:

$$\alpha = \frac{K\,l^4}{E\,I\,\pi^4}\,;\ \gamma = \text{ratio of } \frac{q\,l^2}{8\,h} : \frac{E\,I\,\pi^2}{l^2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (H)$$

By taking for n the values, 1, 3, 5, . . ., the following equations result:

$$\left.\begin{aligned}
&a_1\left[\pi^2 + \alpha\,\pi^2 - \gamma\left(\frac{3}{2}\,\pi^2 - 2\right)\right] \\
&\quad + \gamma\left(\frac{15}{2}\,a_3 + \frac{65}{18}\,a_5 + \frac{175}{72}\,a_7 + \ldots.\right) = 0 \\
&a_1\,\frac{15}{2}\,\gamma + a_3\left[\pi^2\,3^4 + \alpha\,\pi^2 - \gamma\left(\frac{2}{3}\,9\,\pi^2 - 2\right)\right] \\
&\quad + \gamma\left(\frac{255}{8}\,a_5 + \frac{609}{50}\,a_7 + \ldots.\right) = 0 \\
&a_1\,\frac{65}{18}\,\gamma + a_3\,\frac{255}{8}\,\gamma + a_5\left[\pi^2\,5^4 + \alpha\,\pi^2 - \gamma\left(\frac{2}{3}\,5^2\,\pi^2 - 2\right)\right] \\
&\quad + \gamma\left(\frac{1295}{18}\,a_7 + \ldots.\right) = U \\
&a_1\,\frac{175}{72}\,\gamma + a_3\,\frac{609}{50}\,\gamma + a_5\,\frac{1295}{18}\,\gamma \\
&+ a_7\left[\pi^2\,7^4 + \alpha\,\pi^2 - \gamma\left(\frac{2}{3}\,7^2\,\pi^2 - 2\right)\right] + \ldots. = 0
\end{aligned}\right\} \ldots\ldots\ldots (I)$$

Only the coefficients, a_n, with odd subscripts enter into Equations (I) and, as seen from Equation (C), only deflection curves symmetrical with respect to the middle of the span correspond to these equations.

By taking for n the consecutive even values, 2, 4, 6, . . ., the following equations are found:

$$\left.\begin{aligned}
&a_2\left[\pi^2\,2^4 + \alpha\,\pi^2 - \gamma\left(\frac{2}{3}\,\pi^2\,2^2 - 2\right)\right] \\
&\quad + \gamma\left(\frac{160}{9}\,a_4 + \frac{15}{2}\,a_6 + \ldots.\right) = 0 \\
&a_2\,\frac{160}{9}\,\gamma + a_4\left[\pi^2\,4^4 + \alpha\,\pi^2 - \gamma\left(\frac{2}{3}\,\pi^2\,4^2 - 2\right)\right] \\
&\quad + \gamma\left(\frac{1248}{25}\,a_6 + \ldots.\right) = 0 \\
&a_2\,\frac{15}{2}\,\gamma + a_4\,\frac{1248}{25}\,\gamma + a_6\left[\pi^2\,6^4 + \alpha\,\pi^2\right. \\
&\quad \left. - \gamma\left(\frac{2}{3}\,\pi^2\,6^2 - 2\right)\right] + \ldots. = 0
\end{aligned}\right\} \ldots\ldots\ldots (J)$$

Equations (J) contain only the coefficients, a_n, with even subscripts; hence, the corresponding deflection curves have an inflection point at the middle.

It is clear that in order for buckling to occur, at least one or more of the values of a_n given by Equations (I) or (J) must be different from zero. This condition will require that the determinant of Equations (I) or (J) become

equal to zero. The critical values of the compressive force may now be calculated by the following method of successive approximations.

Begin with the case of a very small K; then, as found from the discussion of the simple case in Part III, the deflection curve consists of one wave only and the system of Equations (I) must be used for the calculation of the critical compressive forces. Take, for instance, $K = 0$. Making all the coefficients, a_n, except a_1, equal to zero and retaining only the first equation of the system, (I),

$$\pi^2 - \gamma \left(\frac{2}{3}\pi^2 - 2\right) = 0, \text{ or } \gamma = \frac{\pi^2}{\frac{2}{3}\pi^2 - 2} = 2.15$$

This first approximation, equivalent to the assumption that the deflection curve is a sine wave, is somewhat greater than the more accurate value, $\gamma = 2.06$ given in Table 2. The error of the first approximation is about $4\frac{1}{2}$ per cent.

In order to get a better approximation take two consecutive coefficients, a_1 and a_3, and put the remaining coefficients equal to zero, in the system of Equations (I). Then, the first two equations of this system become:

$$a_1\left[\pi^2 - \gamma\left(\frac{2}{3}\pi^2 - 2\right)\right] + a_3\frac{15}{2}\gamma = 0$$

$$a_1\frac{15}{2}\gamma + a_3\left[\pi^2 3^4 - \gamma\left(\frac{2}{3}\pi^2 9 - 2\right)\right] = 0$$

These equations will yield for a_1 and a_3 a solution different from zero and buckling will occur only when the following condition is fulfilled:

$$\left[\pi^2 - \gamma\left(\frac{2}{3}\pi^2 - 2\right)\right]\left[\pi^2 3^4 - \gamma\left(\frac{2}{3}\pi^2 9 - 2\right)\right] - \left(\frac{15}{2}\gamma\right)^2 = 0$$

Solving this quadratic equation, $\gamma = 2.06$. This second approximation is the value given in Table 2. Calculation of the third approximation shows that in this case the second approximation gives three correct figures for the factor, γ.

In the same manner, calculations were made for,

$$\frac{K l^4}{16 E I} = 5; \quad \frac{K l^4}{16 E I} = 10; \quad \frac{K l^4}{16 E I} = 15$$

and the corresponding values of γ are given in Table 2. For larger values of $\frac{K l^4}{16 E I}$, Equations (J) must be used, because in such cases the deflection curve will have two waves with an inflection point at the middle. With further increase of K curves will be obtained with three, four, and more waves. It is clear that for an odd number of waves, Equations (I), and for an even number, Equations (J), must be used in calculating the critical loads.

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PAPERS AND DISCUSSIONS

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THE SURVEYING AND MAPPING ACTIVITIES OF THE FEDERAL GOVERNMENT*

By T. W. Norcross,† Assoc. M. Am. Soc. C. E.

The Federal Government does more surveying and makes more maps than any other organization, public or private, in the United States. Many among the map-using pubic are well acquainted with the work of one or two Federal agencies, but do not know that many agencies make surveys and that the number of map makers and map publishers is even larger.

Board of Surveys and Maps

The Federal Board of Surveys and Maps is composed of a representative from each of the principal map-making agencies. Its duty is to co-ordinate and standardize all Federal surveying and mapping functions and harmonize these with similar non-Federal activities.

During the World War, and immediately thereafter, it was believed that considerable duplication of work existed and that by consolidation or co-ordination greater efficiency in mapping could be secured. Previously, the U. S. Geographic Board had been given advisory power with the various Federal agencies to avoid unnecessary duplication of work and to unify and improve the scales and details of maps. However, it was very evident that with a few exceptions, these agencies had little or no knowledge of similar work done, under way, or proposed by the others.

On July 1, 1919, the Chairman of Engineering Council addressed a letter to the President, suggesting that he appoint:

"* * * a 'Board of Surveys and Maps' consisting of a representative from each of the present Federal map-making agencies, together with representatives from well-qualified map-using agencies, which would be vested with authority to work out a plan of standardization and co-ordination of the work of the Government, and to report the same to the President for such action as he may find wise."

Note.—Written discussion on this paper will be closed in August, 1929.
* Presented at the meeting of the Surveying and Mapping Division, Washington, D. C., April 26, 1928.
† Chf. Engr., U. S. Forest Service, Washington, D. C.

President Wilson, on August 27, 1919, requested twelve Federal agencies to confer and report to him on the subject. Later, the Forest Service and the U. S. Hydrographic Office were added. In their report, the representatives stated that there was little actual duplication of effort materially affecting the progress of the standard U. S. topographic map; that the U. S. Geological Survey was responsible for the preparation of this map; and that it was co-operating closely with the U. S. Coast and Geodetic Survey, which, in addition to its other work, was engaged in the execution of the required primary control. The early completion of the standard topographic maps was considered of extreme importance.

The conferees unanimously agreed that it is impracticable to make the U. S. topographic map on a scale sufficiently large to meet all Governmental and private needs and adaptable to all special and unusual purposes. Where the standard map does not meet the needs, special surveys would have to be made. While the various map-making agencies had shown a commendable spirit of co-operation, it was the opinion of the conferees that many opportunities for co-ordination existed. Therefore, they recommended that a permanent Board of Surveys and Maps be appointed.

On December 30, 1919, the President promulgated the following Executive Order:

"In order to co-ordinate the activities of the various map-making agencies of the Executive Departments of the Government, to standardize results, and to avoid unnecessary duplication of work, I hereby constitute a Board of Surveys and Maps, to be composed of one representative of each of the following organizations:

"1.—Corps of Engineers, U. S. Army,
2.—U. S. Coast and Geodetic Survey, Department of Commerce,
3.—U. S. Geological Survey, Department of the Interior,
4.—General Land Office, Department of the Interior,
5.—Topography Branch, Post Office Department,
6.—Bureau of Soils, Department of Agriculture,
7.—U. S. Reclamation Service, Department of the Interior,
8.—Bureau of Public Roads, Department of Agriculture,
9.—Bureau of Indian Affairs, Department of the Interior,
10.—Mississippi River Commission, War Department,
11.—U. S. Lake Survey, War Department,
12.—International (Canadian) Boundary Commission, Department of State,
13.—Forest Service, Department of Agriculture,
14.—U. S. Hydrographic Office, Navy Department."

To the fourteen original member organizations the President has since added eight agencies, namely,

Military Intelligence Division, General Staff, War Department,
Federal Power Commission,
Air Corps, War Department,
Bureau of Aeronautics, Navy Department,
Aeronautics Branch, Department of Commerce,
Bureau of Foreign and Domestic Commerce, Department of Commerce,
Geographic Section, Department of State,
Division of Maps, Library of Congress.

This Board, on which each of the twenty-two organizations has a designated representative and an alternate, may act on its own motion or on the request of any organization or individual. The policy is to invite full co-operation from any Federal or non-Federal map-making or map-using agency, by the presentation of its views to the Board and by representation on committees considering matters of interest to such agency. The Standing Committees are: Executive; Co-ordination; Technical Standards; Topographic Maps; General Maps; Hydrographic Charts; Control; Photographic Surveying; Information; Cadastral Surveys; Map Reproduction; Military Maps; and Aerial Navigation Maps. The membership of committees is not restricted to members or alternates of the Board, the intention being to include those Federal employees who are best qualified to advise on the subjects treated and also to give to each bureau engaged in any specific phase of surveying or mapping, representation on the committee handling that work. The Board holds monthly meetings.

To further the purposes of the Board, an Advisory Council was created. This consists of representatives of non-Federal organizations primarily engaged or interested in map-making, as follows:

American Association of State Highway Officials,
American Automobile Association,
American Forestry Association,
American Geographical Society,
American Institute of Electrical Engineers,
American Institute of Mining and Metallurgical Engineers,
American Railway Engineering Association,
American Society of Agricultural Engineers,
American Society of Civil Engineers,
American Society of Mechanical Engineers,
American Water Works Association,
Appalachian Mountain Club,
Association of American Geographers,
Association of State Geologists,
American Engineering Council,
Geological Society of America,
National Electric Light Association,
Rocky Mountain Club,
National Research Council,
Sierra Club,
Society of Automotive Engineers,
Map Publishers,
Society of American Military Engineers.

This Advisory Council is also organized. It is usually represented at the Board meetings by several members and is entitled to have members on practically all the committees with power equal to that of members designated by the Board.

The central information office required by the Executive Order has been established since January, 1920, and is known as the "Map Information Office." As an experiment in co-ordinating closely related work as against combination under one administrative head, the Board of Surveys and Maps has been a decided success. It was soon evident that the duplication in work

and the opportunities to reduce the cost of handling were far less in amount than had previously been believed. Many opportunities, however, were found for better co-ordination of effort or greater progress, and the results accruing from the Board have exceeded the anticipation of those advocating its creation. Much has been done to clarify the mapping situation within the Government and throughout the country.

Many practices and specifications have been established. It is interesting to note that although the Board has not insisted upon compliance with its standards by the member organizations, these standards are generally accepted and followed. Notable among the approved Committee reports are those on vertical and horizontal control for map use or geodetic investigations; on photographic surveying; on standard map symbols; on scale of standard topographic maps; on standards for base maps of the United States; and on use of aerial photographs in topographic mapping.

The Board has made known the methods, plans of operation, and results of each Bureau to all the others, so that full and prompt use can be made of the results. It has brought about uniformity in scales, symbols, and types of projections for maps, and has almost entirely eliminated the possibility for duplication of work.

While opportunities for standardization, co-ordination, and service are continually arising, the Board has already completed the major part of its assigned work. With work now under way executed in accordance with the approved standards, efficient handling and satisfaction to the map users are assured. When appropriations by Congress make possible prosecution of the standard topographic map of the United States on the expenditure scale and also within the 20-year period approved by the Temple Act, the agencies responsible for this mapping can proceed with every assurance that their work will be in accord with specifications and practices that have been found best. All other bureaus whose need of maps exceeds the present rate of progress are now prepared for the larger scale of operations.

Primary Surveying Agencies

Many member organizations of the Board are map makers only to a minor extent, but they use maps extensively. Others do little or no surveying work and base the map preparation on compilation from other maps. Such maps as are made are designed to meet a special need of the bureau.

"Which are the primary surveying and map-making agencies of the Federal Government", is a question frequently raised. Surveying and mapping like other phases of engineering is, of course, a service or a means to an end rather than the end itself. Considering "primary" to refer to those agencies in which surveying and mapping is the major objective, it appears that only one can be so classed and its work is practically completed. Of the member organizations of the Board of Surveys and Maps, a maximum of eight have engineering accomplishments in varying forms, as the major objective, the surveying and mapping work being incidental. The major function of at least four member organizations engaged in surveying, is non-engineering in character. This number would be about trebled by including the member

organizations which do little or no surveying. Interpreting "primary" as relating to the volume of surveying work, the agencies which should be included are the Geological Survey, Coast and Geodetic Survey, General Land Office, Corps of Engineers, and the Lake Survey.

The Federal surveying activities may be classified into Geodetic, Topographic, Location and Construction, Hydrographic, and Cadastral.

Geodetic Surveying.—The geodetic work is largely confined to the Coast and Geodetic Survey, Geological Survey, Hydrographic Office, Corps of Engineers, Lake Survey, Mississippi River Commission, and International Boundary Commission.

The Coast and Geodetic Survey is responsible for the extension of first-order triangulation, first-order traverse, and first-order leveling over the United States. This work is 37% complete. Based on the present (1928) scale of appropriations and progress, it is estimated that approximately fifty years will be required to complete the work.

The control work of the Geological Survey is confined to that required for the preparation of the standard topographic maps; usually this means the third orders of triangulation and leveling. The control work previously done by the Lake Survey was of the first order. The work of the Corps of Engineers and the Mississippi River Commission is largely of the second and third order. The International Boundary Commission did work of the second and third order, its requirements for first-order control being met by the Coast and Geodetic Survey and by the Geodetic Survey of Canada. The Hydrographic Office does such triangulation work as is necessary in making charts prepared from its own surveys.

Topographic Surveying.—Topographic surveying work is carried on by a number of agencies.

The Geological Survey is engaged in making a map of the United States and Alaska, involving both topographic and geologic details. It is the agency recognized as responsible for the standard topographic map of the United States, based on the fundamental control furnished by the Coast and Geodetic Survey. It also conducts investigations relative to surface and underground waters, classifies the public land, and supervises the engineering phases of mineral leasing. At present, approximately 43% of the Continental United States is covered by the topographic map, but about 30% of this must be worked over, corrected, or supplemented. To a great extent the topographic mapping is done in co-operation with the States, but a comparatively small amount of mapping of Federal lands is done wholly at Federal expense. Based on the 1928 scale of expenditures and progress, it is estimated that the topographic map of the United States will be completed in about 90 years, but including the re-surveys that must be made, the total time will approximate 100 years.

The Bureau of Chemistry and Soils is organized under three major lines: (1) Chemical and technological research; (2) soil investigations; and (3) fertilizer and fixed-nitrogen investigations. Under "soil investigations" is grouped all the soil work, including the classification and mapping of the

soils in the United States, studies of the agricultural value, and related subjects, including soil erosion. The desired data on soils for about one-third of the United States have already been secured. The traverse work is done by plane-table and is simple in character. The field men of the Bureau do this work in connection with the mapping of soils. The maps are compiled in Washington, D. C.

The Forest Service administers the National Forests; co-operates with States in maintaining organized protection of forest lands against fire; conducts investigations in the entire field of forestry, including economic and industrial as well as forest research; diffuses information relating to the best use of forests and forest products; promulgates rules and regulations for the administration of grazing on National Forest ranges; and, in short, seeks to promote the application of forestry on both public and privately owned lands, together with efficient and economical use of their yields. In general, the map needs of the Bureau in connection with administering the forests and for protection against fire will be adequately met by the standard U. S. topographic maps and the township plats of the General Land Office. Roughly, 75% of the latter work has been completed and, on the whole, the urgent needs are met fairly well. The topographic map situation is decidedly poorer, only 60% of the forest area having been mapped and about one-third of these maps need correction or additional work. The need of more and better maps is urgent.

All mapping done directly by the Forest Service is incidental to some administrative function. Due to lack of standard topographic maps, drainage surveys are made. In the aggregate, a rather large amount of work is done in locating roads and trails, but this is extremely simple in character. The larger portion of the other mapping is incidental to the collection of information on timber cover and grazing classification.

The military activities of the Corps of Engineers comprise reconnoitering and surveying, together with fortification construction, repairs, etc. The plans contemplate mapping, either independently or by the Geological Survey, large areas on the coast lines and international boundaries. The topographic section in each engineer regiment and varying details of other troops, together with a small force of draftsmen, are available for mapping work.

The function of the International Boundary Commission is to lay down the boundary line in accordance with existing treaties between Alaska and Canada and between the Continental United States and Canada. All the field work has been completed and the necessary map making and printing is nearly done.

The primary function of the Air Corps, so far as mapping and surveying is concerned, is the making of aerial photographs. Until recently and acting under a co-operative arrangement with the Geological Survey, the Air Corps did all such work as was required in the preparation or correction of the standard topographic maps. This work has increased greatly during the past few years and is now of large proportions. The prospects are favorable for still further increase in the amount of air mapping.

The Bureau of Aeronautics in the Navy Department has annually done some airplane photography work for the Geological Survey, including a large amount in Southeastern Alaska.

Location and Construction Surveys.—The Bureau of Reclamation is charged with the investigation, construction, and management of irrigation developments in the arid States and also investigations of reclamation and farm development outside the arid region. It makes examinations of, and surveys for, irrigation works and their location, construction, operation, and maintenance.

The Office of Indian Affairs has charge of the Indian tribes of the United States (exclusive of Alaska), their education, lands, monies, schools, general welfare, and the purchase of supplies. Maps of the reservations are compiled and surveys of irrigation, drainage, and power projects are made as funds are available and necessity demands.

The Bureau of Public Roads supervises the road work done by the States under the provisions of the Federal Highway Act. It co-operates with the National Park Service, Forest Service, and other Bureaus in the survey and construction of certain roads; studies and distributes information on the economics and management of road work; and handles engineering problems relative to farm irrigation, drainage, and structures. Its surveying work is confined largely to the leveling and traverse work required for road location and construction.

The civil activities of the Corps of Engineers consist largely of the execution of projects for the improvement of rivers, harbors, and other navigable waters of the United States. The work also includes the supervision of structures erected in navigable waters. Surveys of rivers and harbors are made in the amount and detail required for the location, construction, and other work. Triangulation and leveling have been extended along portions of the Ohio, Tennessee, Mississippi, and Sacramento Rivers, and in some other localities.

The Mississippi River Commission makes investigations and surveys of the Mississippi River incidental to its main purpose of permanently locating and deepening the channel, preventing destructive floods, protecting the banks, and affording safety to navigation.

Hydrographic Surveying.—The U. S. Coast and Geodetic Survey is charged with the survey of waters and with the publication of charts. This work includes base measures, triangulation, topography, and hydrography along the coasts; the survey of rivers to the head of tide-water or ship navigation; deep-sea soundings; temperature, and current observations along the coasts and throughout the Gulf and Japan streams; magnetic observations and researches and the publication of maps showing the variations of terrestrial magnetism; seismological investigations; gravity research; determination of heights; and the determination of geographic positions by astronomic observations for latitude, longitude, and azimuth, and by triangulation, to furnish reference points for State surveys. Charts are available for the entire sea coast of the Continental United States but, due to the increased size of ships and to changes due to Man and Nature, the charts for only about 65% of the entire coast are considered adequate. Charts are available

for about 13% of the coast of Alaska, if Bering Sea is included, but the coast which is important to commerce and industry may be considered as about half charted. The work in the Philippines is 80 to 85% completed. On this work substantial co-operation was given by the Philippine Government. The work for the possessions in the West Indies and for the Panama Canal may be considered to be complete. The charts for Hawaii are about half done.

The primary purpose of the U. S. Lake Survey is the surveying and charting, in the interests of navigation, of the waters and shores of the Great Lakes, connecting and outflowing rivers, Lake Champlain, the natural navigable waters of the New York canals, Lake of the Woods, and other boundary and connecting waters between that lake and Lake Superior. The work is similar in kind and purpose to that done by the Coast and Geodetic Survey along the seaboard. The program of surveys on the Great Lakes is about 95% complete. The surveys on the New York canals have been finished and the work on the Lake of the Woods and adjacent waters is about 35% done. Considerable re-survey work of Lake Champlain will be necessary to modernize the charts.

The U. S. Hydrographic Office compiles and publishes charts of foreign waters from data obtained from available sources. It collects and disseminates hydrographic and navigation data, provides charts, sailing directions, and manuals for the use of the U. S. Navy and for navigators. Charts are now available for about five-sevenths of the surveyed world outside the coasts under the jurisdiction of the United States.

Cadastral Surveys.—The principal agency engaged in cadastral surveying is the General Land Office, this work being incidental to the survey, management, and disposition of the public lands, the adjudication of conflicting claims relating thereto, and the granting of railroad and other rights of way, easements, and the issuance of patents for lands. During the fiscal year ending June 30, 1927, almost 4 000 000 acres were surveyed and about 1 200 000 acres more were re-surveyed. The total acreage surveyed to that date was almost 1 300 000 000. The job of surveying and subdividing the public lands of the United States, including Alaska, is now about 71% completed, but with the vast amount of re-survey work that is constantly being demanded, it is impossible to foresee when the work will come to an end. The mapping work is usually on a drainage basis and is done in connection with the regular township survey work. Topography is accurately secured on the surveyed section lines, but the interior of the section is usually sketched in. Relief is ordinarily shown by hachure.

The activities of the Forest Service in cadastral surveying are practically confined to surveys incidental to the purchase of land to be included in the National Forests. These surveys are located almost entirely in the eastern part of the United States.

Map-Making and Reproduction

From the standpoint of the volume of maps of different areas made and published, the Topographic Branch of the Geological Survey undoubtedly leads, but many maps are made by others, particularly the Hydrographic Office,

Corps of Engineers, Coast and Geodetic Survey, General Land Office, Topographic Branch of the Post Office Department, Bureau of Soils, Forest Service, Bureau of Public Roads, U. S. Lake Survey, International Boundary Commission, Mississippi River Commission, and Military Intelligence Division of the General Staff, War Department.

Maps of the United States are published by the Geological Survey, General Land Office, Coast and Geodetic Survey, Forest Service, and Bureau of Public Roads. No two maps contain exactly the same data. For general information maps, those published by the General Land Office and the Geological Survey are usually the most serviceable. The maps of the Bureau of Public Roads are essentially road maps, while the primary reason for the Forest Service map is to show the National Forests and other Federal reservations.

State maps are prepared and published by the Geological Survey (scales of 1 : 500 000 and 1 : 1 000 000), Topographic Division of the Post Office Department, Bureau of Public Roads, General Land Office (Public Land States only), and Forest Service (a few of the National Forest States).

General maps and special maps and charts are published, as follows:

Forest Service: Each National Forest, 4 miles to 1 in.; certain National Forests, 2 miles to 1 in.; a few National Forests, 1 mile to 1 in.

Bureau of Soils: Each soil survey.

Coast and Geodetic Survey: Hydrographic charts of coast lines and all harbors.

Geological Survey: Standard topographic quadrangle sheets on the scale of 1 : 62 500, 1 : 125 000, or 1 : 250 000; special topographic maps on scales larger than the standard; international map on scale of 1 : 1 000 000; geologic maps.

General Land Office: Township plats; mineral plats; re-survey plats of townships; supplemental diagram plats.

Reclamation Survey: Project maps.

U. S. Hydrographic Survey: General maps of the world; general ocean charts; general coast charts; pilot charts; coast charts; and harbor charts.

Topographic Division of Post Office Department: County maps showing rural delivery routes; local center maps.

Corps of Engineers (Civil Activities): Detail maps of projects.

Corps of Engineers (Military Activities): Sectional sheets of progressive military maps of the United States; special maps of military reservations and other areas strategically important from a military standpoint.

Lake Survey: General charts of Great Lakes; general lake charts similar to that of Lake Erie; charts of lake shore line; and harbor charts.

Mississippi River Commission: Charts of the Mississippi; various special maps on comparatively large scale, for instance, maps of St. Francis Basin.

International Boundary Commission: Strip maps of areas along the Alaska boundary and the United States-Canada boundary; maps of sections along Canada-United States boundary.

Map Information

With so many Federal agencies making maps, it is not at all surprising that any one desiring a map giving special information or for a certain area

is sometimes bewildered and does not know where to make application. To obviate this difficulty, it was even suggested that all map-making agencies be combined under one administrative head. The difficulty was satisfactorily removed, however, without affecting bureau operations or separating engineering functions from the administrative functions which they directly serve.

In accordance with the Executive Order of December 30, 1919, the Map Information Office of the Board of Surveys and Maps was created. It is located in the Interior Department Building. This Office acts as a clearing house for special topographic, hydrographic, and geodetic information as well as for that of a general geographic character. If the inquirer knows the agency the duty of which it is to obtain the desired information, he naturally sends his request direct; otherwise, he seeks the aid of the Map Information Office, which has direct communication with all the Federal offices from which the desired information can be readily obtained.

Additional to information derived from Federal sources, the office has on file catalogues and reference books of various kinds from commercial sources as well as from foreign countries. The Map Information Office does not carry a stock of maps for distribution and has not attempted building up a complete file of all published maps. One copy of each type of map is available for display or reference. If a map is needed, it is advisable to ask the Map Information Office. Frequently, duplication of work will be avoided thereby; maps unnecessarily made are an unjustified burden on those who first or last pay the bill.

The Federal agencies are the servants of the public. Due to their map information and experience in surveying and map-making they can render valuable aid. If information or advice is desired they are glad to be of assistance. Every effort to give help will be made.

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GOOD MAPS ARE A SOUND INVESTMENT*

BY L. R. LOHR,† M. AM. SOC. C. E.

SYNOPSIS

Maps permit a nation or a sub-division thereof to take stock of its assets. It is impossible to inventory natural resources intelligently without proper maps to plot their location and extent. On these maps may be indicated the soil classification of the land surveys, what sections are most suitable for agriculture, grazing, and timber, and what sections should be set aside for clearing and for reforestation. They make possible a knowledge of the extent of water power available for conversion into electrical energy, the quantity that may be used for irrigation and for drinking. They will tell what swamp land should be drained; what streams, rivers, and lakes are navigable; the best routes on them, and the hazards to navigation. They will show the best location for roads and railroads—the most direct routes with the lowest grades at the lowest cost per ton-mile. They show the geological formations which are guides to mineral resources, including the important reserves of coal and oil. For engineering operations, they show the elevation of every point on the land and water and the distances between locations.

They enable the traveler to find his way; engineers and promotors to decide whether plans are feasible, and to calculate their cost. With a complete system of maps, interest in scientific prospecting and mining will be stimulated and facilitated.

It is not sufficient for an engineer to know the importance of a map, he also must be able to present to his employers such concrete and well-founded facts that they may comprehend and accept them. In this age of dollars and cents, it will take definite figures to swing a necessarily materialistic board of directors or city council to a full appreciation of the value of maps. The figures are available, but ordinarily are little known.

NOTE.—Written discussion on this paper will be closed in August, 1929.

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Maps, in their broadest sense, are so interwoven with the fundamentals of modern institutions that they have become an integral part of the entire economic structure. In all ages, from the time that the Cro-Magnon man scratched his rude charts on his cave in the Pyrenees, through the epoch-making military achievements of Xerxes, Alexander, and Cæsar, through the explorations of Captain John Smith who mapped Chesapeake Bay 300 years ago, down to the great construction projects of modern times, maps have been an important factor in making human history. Facts constitute the foundations on which every engineering structure should be built. Since maps supply in graphic form the facts of terrain they are the ultimate basis of practically all engineering and industrial work.

The early Nineteenth Century development was largely in the hands of pioneers, fired with the Anglo-Saxon desire for activity and immediate results. There was no time for elaborate surveys; railroads had to be built, public lands had to be subdivided and tenanted. Communities sprang into existence and flourished with a rapidity amazing to the more leisurely and deliberate Europeans. During this epoch, there was little demand for maps, not only because of the time factor, but because the maps and methods of the day were crude.

When the feverish activity of building settled to a steady routine, when expensive relocations became desirable, and when the Courts became clogged with boundary disputes, a few people began to realize that building had been backward, that the transit should have followed the covered wagon and the rifle. Desirability, however, does not always create a demand—modern advertising expenditures prove that. With the handwriting of sad experience on every wall, the Twentieth Century finds still only a limited demand for maps, as is evidenced by the fact that the Temple Act which provides mapping the entire United States on a scale of 1: 62 500, and within 20 years, has lain dormant for several years since its passage.

Does this generally apathetic attitude mean that maps are not worth the cost? Rather, it means that their full economic value is not commonly understood and that there has been no systematic plan of education. As a rule, map makers have been so absorbed in their own technical problems that the voice of the few who have given thought to the economic side has been but a weak wail in the wilderness.

Volumes have been written on every technical phase of map making, but almost nothing on map using. Engineers' efforts have been one-sided. They have been content with perfecting a desirable article, with the attitude that, if the public cannot see its utility, no one is to blame but itself. This apathy may be because most mapping has been done by governmental agencies, the primary function of which is that of production.

However, light is beginning to dawn, as is shown by the formation of the Society's Surveying and Mapping Division and of the Federal Board of Surveys and Maps and by the inclusion of more comprehensive courses on the subject in university curricula. When the economics of maps are more generally understood, the greater demand will bring a larger supply. It is true that the utility of the topographic map has long been established, but so was

the utility of gun-powder centuries before it was used efficiently. The need of good maps is known, but as Mark Twain said about the bad weather, "nobody does anything about it".

Presenting Map Values to Laymen

In recent years, the greatest demand for high grade maps has been, not in frontier regions into which civilization is pushing its way, but in the more densely settled regions which may have been mapped repeatedly. This is because business customarily withholds all expenditure until conditions justify it. In its early development a corporation is reluctant to sink money in map making, because it is not a tangible asset on which a loan in the form of stock or bonds can be floated.

In any industrial development it is the board of directors that decides on expenditures. To it maps may seem an intangible luxury instead of the difference between an economic guess and an economic certainty. An automatic machine which will double the Company's production, or a salesman who can increase sales, has attributes more easily grasped. An engineer may know that maps are as necessary to him as instruments to a surgeon, but when a layman inquires of him specifically how valuable are the maps the answer is likely to be unconvincing. He has considered them to be so essential that he has not attempted to define their economic advantages.

It is not sufficient for an engineer to know the importance of a map, he also must be able to present to his employers such concrete and well-founded facts that they may comprehend and accept them. Comparative cost data are seldom available because, for instance, two dams are not built on the identical location, one with and one without maps.

The Map as a Base

Because Man's mind is incapable of grasping, holding, and manipulating all the facts of Nature, he makes use of mathematics to develop formulas, and of drawings to depict plans. In this way one obstacle at a time is overcome and set aside as reference, where it is immediately available and can be brought back into the picture when needed. A mechanical engineer would not consider planning in his head all the parts of a complicated machine and then proceeding with its construction. Likewise, it is impossible for a civil engineer to hold in his mind the multitudinous facts regarding a piece of ground.

Complete and accurate topographic information brings to an engineer designing a project all the physical data in the most usable form, so that the different parts may be studied in their relationship to the whole. It allows a study of an infinite variety of alternatives, provides for any number of trial plans, and greatly increases the probability of securing the one most economical location. If the survey is properly controlled and monumented, it is a simple matter to make a comprehensive design and transfer it from the paper to the ground.

It is seldom economical, however, to make a topographic map of such precision that a project of any magnitude may be given a final location. From the map, it may be laid out within close enough limits for guidance of the field men.

United States Geological Survey Sheets

The United States is about 43% mapped, mostly by the U. S. Geological Survey on a scale of 1 : 62 500, although the maps of nearly one-third of the area covered are out of date. These sheets form a general utility map and are excellent for most forms of preliminary surveys—for highway locations, for sites for hydro-electric developments, for reclamation projects, etc. Such a general-purpose map, however, will not meet all needs for preliminary reconnaissance. For example, in the case of the Catskill Aqueduct it was necessary to run out the locations in the field, because geological structure was of more determining influence than surface relief.

In designing a certain trunk sewer an estimate was made on a Geological Survey map. Some question arose as to the correct location of a divide; a single line of traverse run through the area showed nearly double the acreage used in the run-off computations. It was not that the map was unreliable, but that it was being used for a purpose for which it was not intended. The making of large-scale maps for local areas will always be necessary for special purposes.

Poor Maps, a False Economy

Poor maps are frequently as bad or worse than no maps. Not only may they be misleading in a vital item, but they actually stand in the way of an accurate survey.

As most maps are reasonably authentic, the tendency is to accept without question the data printed thereon. Because a location is definitely shown on a map does not prove that it is correct, as many costly mistakes will testify. Each map should be scrutinized or tested for its accuracy. If it is a Government map, it may bear the date of its making, and this is a good indication of what may be expected of it; if its source is unknown, sufficient lines should be run in the field to establish its reliability. Where design is particularly dependent on map data it may be well to check the details. The rare case where a mistake is found will justify the procedure as in a certain hydro-electric construction already well under way before a new survey proved the original one so inaccurate that the entire project had to be abandoned.

The exactness and detail necessary for any particular map are questions for which solutions must be based on experience and good judgment. As a rule this should be determined by the engineer responsible for the plan and not by the surveyor. As designer, the engineer will know the limits of tolerance, and, therefore, may specify the scale on which he desires the survey made. Since, in properly made maps, the accuracy depends on the scale, he has a definite index for his calculations. What the scale shall be is usually determined by rule of thumb. For reclamation maps, 5-ft. contour intervals were found to be insufficient, and it was necessary to resort to those of 1 ft. or 2 ft.

Immense sums are spent annually for emergency surveys executed with no thought and little chance of finding uses other than the single one for which they were made. They are seldom controlled by triangulation or referred to a datum, with the result that subsequent users have little confidence in the work. It is to be hoped that the United States control net will shortly cover such a

large percentage of the country that even a small private survey may start from a definite reference point and hence the ensuing work be of more permanent value. Equally important is the question of monumentation. Even emergency surveys should be permanently marked on the ground, and notes and reference points accurately made for future use.

With the fully controlled topographic map of the United States which the Temple Act would give, there would be available a base for any map requiring a lesser degree of refinement. Hundreds of surveys are made annually, such as for automobile road maps, which, as a rule, are valueless for any other purpose. With the complete atlas all such surveys could be eliminated.

A frequent cause of waste is that the area included is not sufficiently large. This is particularly true in municipalities. A map is made of an area in which improvements, such as sewer lines are projected, and is satisfactory for that purpose; but when the extensions inevitably come and new sewer connections are made it is found that the original layout was not designed to take care of the additions. The cost of relocating a large part of a sewer system has involved more of the taxpayers' money than to have made a map of the entire area in the beginning.

For many years the State Reclamation Service of Texas has demonstrated the true value of accurate maps. These are made on the scale of 1 000 ft. to the inch with a 2-ft. contour interval. The completed map is brought to the office at Austin, where all the problems in levee design are worked out and all hydraulic and earthwork calculations made. The levees and drains are actually located on this topographic map in the office, perhaps 100 miles from where the improvements are to be built, as well as the proper lines and grades. High-water levels are determined precisely, as are total discharges, preliminary quantities, areas of land expected to be damaged, and even the final development of the protected land along agricultural lines.

The Austin Office reports that, in the many years during which this perfected plan has been in use, there never has been a mistake or failure in the procedure. Without a map of the high degree of accuracy needed in this work, the whole system would have been doomed to failure.

It is the irony of fate, however, that the more refined and detailed a map, the sooner it is out of date. With a good base available, however, revision may be made readily and cheaply by the use of aerial photographs, as has been demonstrated in the Los Angeles Quadrangle.

Preliminary Investigation of Water Resources

One of the most valuable uses of the general utility map is in connection with the preliminary reconnaissance of water resources, which includes irrigation, municipal supply, and power development. These projects frequently cover many square miles, for, in order to determine the run-off, it may be necessary to plot the complete water-shed. In other cases, the water must be conducted by artificial means for long distances. A map having accurate contours is absolutely essential to such work.

Existing accurate maps materially reduce the cost of making reports in connection with hydraulic developments. Areas are thereby developed which

otherwise might have been overlooked or have failed to receive full consideration, due to the fact that no one was willing to gamble to the extent of the expense involved in securing reliable information.

In the studies for the Catskill Water Supply, standard topographic maps from the U. S. Geological Survey were used, saving a heavy cost for preliminary field work. A number of routes which might have appeared to be good locations were eliminated without further consideration by a simple reference to the standard sheets.

To utilize for irrigation and reclamation the flood waters of the San Simon Valley of Arizona, it was found that the standard Geological Survey sheets gave practically all the requisite information and, with some limited surveying on the ground, the whole of this rather elaborate plan, including a dependable cost estimate, was worked out. A complete survey would have been so costly that the interested land owners would probably have failed to raise the amount needed, and the project would have died in its incipiency.

Highways

Many instances are available to show the economic value of maps in highway work. At a cost of more than $5 000, a field party endeavored to locate a highway line between Lafollette and Jellico, Tenn., giving a route more than 26 miles long. Later, a strip map was made of the territory and from it a road was located having a maximum grade of only 6% and being exactly 19 miles long. This effected a saving of 7 miles and $200 000 of expense.

The saving to future generations in operating costs on shorter routes and lesser grades, cannot be estimated. If such large savings were made in short stretches of road, however, it does not put a great tax on the imagination to visualize the colossal sum that would be saved to the Nation in its complete system of highways.

Railroads

During the era immediately before and after the Civil War, there was the great cry of "Westward Ho". With frantic speed, the long ribbons of steel were laid across mountains and prairies. The known gaps in the mountains were few, and the line of the railway was frequently determined by a horseback reconnaissance to verify the accounts of Indians that such and such a pass existed. In most cases, there was no question of balancing the desirability of different routes. With competition keen and a clamoring demand, the overshadowing need was to get the line built.

This was done, and the opening of the West is one of the most glowing epochs of American history. With the advent of reliable maps, however, the railroad companies have been continually straightening their lines and improving their grades. By the examination of an advanced copy of a topographic map, a new location for a proposed short section of the Wabash Railway, near Cadiz, Ohio, saved 2 200 ft. in distance, 75° of curvature, and 600 ft. of tunnel, thereby reducing the expenditure for this part of the road by at least $85 000.

The statement has been made by Gen. Francis A. Walker that "if the State of Massachusetts had had a good topographic map in 1826, some twenty million dollars would have been saved in the public railway expenditure."* Such a saving would amount to more than $2 400 per sq. mile, while a topographic survey would have cost less than $50 per sq. mile. Only a nation of such great natural wealth could have stood such losses. An accurate and controlled topographic map of the entire United States would have cost less than $100 000 000, yet here is one of the smaller States reporting a loss of one-fifth of this total in connection with only one of the many uses to which the map could have been put.

Cadastral Maps and Boundary Surveys

The cadastral maps of Babylon depended on the descriptive fixity of various places on charts and this is used in principle even to-day.

One of the vastest enterprises based by statute on cartographic representation was the disposal of the nation's public lands comprising more than one-half the total area and now supporting 57 000 000 people. Because of inaccurate original property surveys, of monumentation so poor that the marks were soon lost, and of descriptive field notes so carelessly written as to be indeterminate and even contradictory, there has been a vast and unnecessary expenditure of money in legal action, and in re-surveys—not to mention the losses to original land owners or their heirs through the development of voids. Only 10 years ago, during the oil boom in Eastland County, Texas, a surveyor, who was re-running old lines, in a single year found voids that would have been worth more than $100 000. Texas is not a Public Land State, but its lands were surveyed in much the same manner as the Federal public lands.

The question of boundary dispute is as old as land ownership. A famous case of other days was the question of the boundary line between the United States and Mexico, as determined by the Treaty of Guadalupe-Hidalgo in 1848, and laid down on a specified map of the United States, a copy of which was appended to the treaty. When an attempt was made to run this boundary on the ground, it was found that errors of 2 or 3° existed in the projection of the map, so that it was impossible to execute the text of the treaty. The sum of $10 000 000 was paid to the Mexican Government to expiate the ambiguity of the map.

Maps are assuming a more important rôle in State jurisdictions. One of the most difficult cases brought before the Court is when boundary lines are alluvial and meandering streams. The investigation of riparian conditions, to determine the status of lands affected by accretive and avulsive changes in the channels of rivers, requires topographic maps in connection with cadastral surveys. Mapping work of this character is becoming regular procedure in all actions involving a changed status resulting from natural causes, or from material error in the early surveys.

For a number of years, the Federal Government has appreciated the economic value, not only of accurate maps, but of maps which can be used for a variety of purposes. A fair example of the efficiency of such practice is in

* Quoted from a private report on a Geodetic Survey of Canada.

the cadastral-topographic survey of the Motanuska and Bering Rivers and Nenana coal fields in Alaska in 1915, the primary object of which was to identify coal lands, partly for Naval reserve and partly for leasing by definite units to authorized applicants for mining. There were made a cadastral survey by legal sub-division; a topographic survey controlled horizontally; and then a grouping of legal sub-divisions and fractions thereof into practical coal-mining units by considering the topography. Within one year, the field work had been done, the plats had been drawn, and the engineering and cartographic groundwork for the business of leasing the lands in all three fields had been completed. The economic value of this combined survey is readily apparent, and any lesser co-ordination of effort would have entailed a loss in both time and money.

Maps in City Planning

Parallel with the development of "preventive medicine", has been what might be termed the art of "preventive surveying" by which engineers to-day may correct many possible mistakes before the construction work is actually started.

This is particularly true in city planning, which has for its basic object improving the environment of people, and, consequently, their health, intelligence, and morality. Many of the fundamental problems in the life of a community may be solved by the aid of an accurate topographic map. It permits natural topography to be utilized as an asset and not fought as a liability.

An alternative to design on the map is experimental building. On every hand errors of other days are being corrected, streets widened, grades lowered, and even new streets opened through built-up sections. The crazy-quilt pattern of most street plans is eloquent of the lack of forethought in their development, which means that they were not planned with an accurate topographic map as a base.

To-day, a map is essential to zoning, to laying out streets, to designing sewers, to placing parks and playgrounds, and to routing transportation. Cities grow principally by the addition of new sub-divisions. In them, it is important that drainage follow natural lines to existing sewers.

Many sewers actually slope the wrong way, with the consequent flooding of cellars and streets. In a small town near Cincinnati, Ohio, there is a sewer which does not begin to discharge until its contents stand 14 ft. deep in a manhole a block away from the outlet. The effect on community health is obvious.

Many cities are discovering that a large part of the land within their boundaries is not on their tax books. On making a map, in this case by aerial photographs, one city discovered that 25% of its area had not been included in assessment. The increased revenue in taxes from this survey more than paid its cost. As a result of maps, tax rates have been actually cut, and the total taxable value has been increased, with the subsequent acquisition of additional funds for public improvement.

City engineers often change, so that the work accomplished during any one administration is usually only a fragment of the whole plan. An accurate map on which developments have been plotted gives to the incoming engineer definite evidence of the projects and accomplishments of its predecessors.

National Defense

One of the greatest values of a topographic map is to National defense. No one is forced to adapt himself more quickly to his environment than the soldier in active operation. A dependable map not only facilitates this by allowing him to study the field before his arrival, but permits him to orient himself and quickly distribute his forces on the ground.

It is axiomatic that reliable maps of the strategic frontiers are indispensable for the preparation and execution of modern war plans. All operations beyond the most rudimentary troop leading, are worked out on maps during peace-time training, in preparation, of course, for efficient practice in war. The uses for maps are almost innumerable in actual operations. They are used in the fire control of artillery and infantry weapons; in the planning of operations; in the direction of troop movements and concentrations; in the selection of positions; in the locations and layout of defenses and fortifications; in the planning of offensives. In the support of actual operations, maps are used in the location, proper co-ordination and construction of bases, depots, camps, hospitals, lines of communication, and dumps. One of the lasting lessons of the World War was the realization of the importance of adequate maps. They were so essential to the conduct of operations that editions showing important areas were demanded every day.

Practically all European countries are completely and precisely mapped. This has been done almost entirely by, or for, the military authorities. The map which is vital to National defense also meets all reasonable demands of peace-time activities. The costs of a modern war are so stupendous that any agency which may shorten its duration for even a day must be considered thoroughly. In the event of an armed invasion of the United States accurate maps might reduce the duration of the struggle by months.

While the soldier must have maps it is of more importance that he be able to read them. His failure to possess maps at a critical time may have inflicted scores of casualties, but the inability to make proper use of maps actually in his possession, has doubtless caused deaths by the hundreds. From a defense standpoint, education in the uses and the reading of maps among the entire public would be an invaluable asset.

Modern field artillery has a power beyond the range of human vision. Maps must be of such accuracy that fire data may be computed with precision. This is particularly important in the firing of barrages where, if the line of exploding shells is too far in front of advancing troops, the enemy has an opportunity to leave its shelters and meet the infantry with machine-gun fire. On the other hand, if the fire is short, it will fall on the attackers themselves with a disastrous effect upon their morale.

The larger caliber guns may fire 20 miles within the enemy lines, but, on account of the short life of the gun, the expenditure of ammunition must be reduced to a minimum. It is essential that there be a large area correctly mapped so that no shell may be needlessly wasted. The value of maps to rapidly developing National air forces, showing routes and emergency landing fields, is too obvious for discussion.

In spite of all this, there are large areas along the American coasts and borders which are entirely unmapped. The standard 1:62 500 map has been adopted for Army use and will serve in all except special localities which will require a scale of 1 to 20 000.

Geology

Topographic maps and geology are inseparable and, where maps are not in existence, the geologist must prepare them as he goes. He simply needs to keep track of his elevation, possibly with only a hand level, and of his position.

Many features shown on his topographic map immediately indicate where he may look for certain geological conditions. Here he finds the key to near-by deposits of economic value. To represent adequately geological formations, a map must have accurate elevations on which can be shown the structure of the rock strata, the thickness of the formation and extent of outcrop, and the amount of cover.

So necessary are maps that one geologist reports that in twelve years of geological activity in the United States he has spent more than $100 000 for maps. In the West, whenever oil geologists take a contract to examine a piece of land for oil prospects, their charges are from 50 to 100% more for unmapped territory than for areas for which there are maps.

Without standard sheets the same territory may be mapped repeatedly. In one particular oil area, half a dozen surveys were made by competing prospectors, each of whom naturally kept his information from the others. The State which has studied its mineral resources and published their descriptions on maps is the one likely to lead in their exploitation and development.

Can Such Things Be?

In view of the actual savings by the use of maps, it seems inconceivable that engineers would permit work to be undertaken without them, yet such instances are numerous.

Any topographer knows the difficulty of determining elevations and slopes merely by eye—that he will occasionally see the apparent phenomena of water running up hill. A real estate man of the old school with his eye and index finger laid out some of the important units of a development, including a short railway spur. His engineer objected and finally persuaded the realtor to permit a survey, which showed the ends of the spur at a difference of elevation of 15 ft. instead of the 3 ft. estimated by the latter. This was sufficient to convince him of the necessity of a contour map.

Other Uses of Maps

Adequate charts are not only essential to navigation but insure against a loss which is inestimable. A pinnacle rock undiscovered or improperly located might send a great vessel to the bottom with the subsequent loss of lives and cargo.

The use of maps is not restricted to engineers and scientists and it is highly probable that, of the million topographic sheets sold annually by the U. S. Geological Survey, many go into other hands. The automobilist uses

them as road guides; legions of campers, mountain climbers, hikers, tourists, and lovers of the great outdoors use them to acquire a keener appreciation of the country through which they ramble.

An enterprising merchant in a small New York town papered a wall with contiguous sheets covering Central New York. It was a source of constant reference by his townspeople. Many who came to consult, stayed to buy.

Conclusion

It is economically more desirable to have general utility maps made by a National agency than by municipalities or private interests. The efforts of the latter are largely sporadic, piecemeal, and uncorrelated. While local co-operation is desirable, it is better that the work be done by the Federal Government, as it is easier to insure uniformity and a higher standard of work and to obtain a greater economy of administration.

With full and authentic map information available, there can be a more intelligent study of natural resources, industrial improvement, and commercial opportunities, and there can be co-ordinated development in accordance with the undisputed facts of map data.

A campaign to educate all engineers to the economic value of maps would be profitable; but this is only going a small part of the way, for, rather than either map maker or map user, it is the financier, the industrialist, and the administrator, who must be converted. They are the mountain to Mahomet; if they will not come to maps, maps must go to them.

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BOUNDARY SURVEYS

PROGRESS REPORT OF THE COMMITTEE OF THE SURVEYING AND MAPPING DIVISION*

By Clarence T. Johnston,† M. Am. Soc. C. E.

In the final analysis, boundaries are mathematical lines limiting the area and form of freeholds, of public and semi-public preserves, and of political sub-divisions. While the coasts of oceans, shores of lakes, and banks of streams may serve for immediate reference, surveys finally determine the boundaries they represent in a general way. National governments frequently sub-divide public land in anticipation of settlement.

The following classification seems to include all surveys made for the purpose of locating boundaries:

Boundary Surveys.—

A.—Cadastral Surveys:

1.—Original public land surveys, domestic and foreign.
2.—Boundaries of lands reserved for public uses, including forest reserves, military reserves, Indian lands, public parks, sites for public buildings and public works, swamp lands, etc.
3.—Boundaries of private claims on unsurveyed land.
4.—Boundaries of lands set aside for canals, reservoirs, railways, highways, drains, electric transmission lines, water-works, and sewer systems.
5.—Mineral land surveys.
6.—Private land surveys:
   (*a*) Retracing public land surveys.
   (*b*) Sub-dividing areas held by freeholders.

B.—Political Boundaries:‡

1.—National boundaries.
2.—State boundaries.

* Presented at the meeting of the Surveying and Mapping Division, Washington, D. C., April 26, 1928. This is a report of the Chairman of the Committee of the Surveying and Mapping Division on Boundary Surveys which consists of Messrs. Clarence T. Johnston, *Chairman*, G. M. Ames, G. H. Fenkell, F. M. Johnson, and H. J. Sherman. The report has not been acted on by the Executive Committee of the Division or by the other members of the Committee.

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‡ The common terms used in the United States are used. Political divisions in foreign countries may vary in detail. Any of these boundaries may be on land, along rivers, lakes, mountain chains, or the ocean, wholly or in part.

3.—County boundaries.
4.—Township boundaries.
5.—Municipal boundaries.

C.—Riparian Boundaries:

1.—Maritime, along sea coasts.
2.—Along navigable rivers or lakes.
3.—Along inland rivers and lakes largely not navigable.

Riparian boundary surveys form the most difficult group. The three general classifications are not defined and merge into each other in a way that baffles analysis. Definition of navigable waters is lacking; also, definitions of the threads of streams, or of the middles of lakes and rivers. Something is given here and there, relative to mean high water, but the general expressions, "mean high-water mark", or "mean low-water mark", are to be found in the law and in regulations, and even in Court decisions. The man in the field, however, who is presumed to set monuments for such marks, has thus far received but little aid from the law or from science.

Cadastral Surveys.—Prior to the Revolutionary War, large areas of lands lying west of the Thirteen Colonies were claimed by the Mother Country. Several Colonies had received something in the nature of concessions which gave them some claim to areas described in very general terms. There were many conflicts between descriptions of lands claimed by the original States, and it was evident to the framers of the Constitution that public lands should be transferred to the Federal Government to avoid trouble between States, to support a unified and progressive land policy, and to afford revenue to a much depleted treasury. The proper administration of a vast expanse of public lands by a democratic people involved responsibilities that seemed to have no precedent in the history of nations. No expert advice was available as the new Government began to function, and there was no marked evidence of special ability for more than fifty years following the signing of the Constitution. During this period public lands were disposed of in large blocks to individuals and companies who, in turn, sold them to settlers. Most of these concessions were made by special acts of Congress. It was necessary for Congress to have some knowledge of the lands in order to avoid over-lapping grants.

After much discussion and experiment, a system of rectangular public land surveys was adopted and was carried on under the wasteful contract system for more than one hundred years. Inspectors of public land surveys were not appointed until toward the end of the Nineteenth Century. The cost of the surveys themselves plus the cost of inspection led to a study of plans looking toward a complete reform. In the meantime, engineering schools had provided scientific training for a few engineers who became interested in work of this kind. Thanks to the influence of this new element and to the good judgment of the Commissioner of the General Land Office, the contract system was abolished during the administration of President Roosevelt. Competent surveyors now give all their time and attention to public land surveys.

In this field of public endeavor, as in many others, the details of legislation and administration appear favorable. The Nation started without a definite plan or purpose in so far as public lands are concerned; but gradually the administration of them has improved. The service could still be bettered by employing qualified men throughout all branches of the land administration. It is certain that improvement rather than retrogression will be recorded as time goes on.

Fortunately, it was unnecessary to determine the boundaries of many areas reserved for parks and similar purposes until after much improvement had taken place in public land surveys. Thus, engineers with the best training became available for this special work. At present, the General Land Office is able to co-operate with several other departments of the Federal Service in which competent surveyors are retained.

In the survey of public lands many private land claims were encountered, some belonging to persons not citizens of the United States. While the Government has been careless in protecting the pioneer, those who occupied lands before the Government could afford any protection have received uniform justice from Congress and from administrative officers. Private land claims of this kind are respected and their boundaries are determined.

While the great area of public lands was passing to private ownership, much other development was in progress. Lines of communication were being extended and connected. As a rule, the long narrow strips of land needed for railways, highways, and canals presented something of a puzzle, particularly to those of legal training. It was necessary to describe them and ultimately to mark their boundaries.

Many people entered California, following the Mexican War and the discovery of gold, either by way of Panama or by overland trails. Most of them intended to engage in mining. Conflicts arose between them relative to titles to mineral lands as well as to the boundaries of claims. They had common traditions and were able to compromise and thus avoid open conflict and serious losses. They met in informal conventions and agreed upon rules which every community would enforce. A Federal Court was the first sign of governmental machinery in this new country. It adopted many of the miners' rules relating to lands and waters.

Congress was soon called upon to enact laws affecting the disposition of mineral lands of the public domain. These laws now embody many of the same rules. The shape and areas of placer and lode claims under the laws of the nation, and the procedure looking to patent, are similar to those accepted by the early miners of California. As mining districts developed, particularly throughout the West, the United States mineral deputy surveyor became a recognized necessity. He is required to pass strict examinations and then survey all claims to mineral lands before patents are granted by the Federal Government.

It requires no great imagination to foresee the time when all public land surveys will have been made, when lands reserved for public purposes will have been laid out and their boundaries defined. Lines of communication and similar developments will continue to be extended, but there never again will

be such a period of activity as during the thirty years following the Civil War. Surveys of mineral lands will continue for a long time.

In contrast with these groups, private land surveys are becoming more complicated and of more importance. Through the operation of public land laws, patents are issued to entrymen. When these are recorded locally, the land described is privately owned and is subject to local taxes. The average value of lands, at the time of patent, was probably not much more than $2 or $3 per acre. Much of it was bought under the Pre-emption Law, or under the commutation clause of the Homestead Act, for about $1.25 per acre. It was considered impossible to pay surveyors very much for establishing boundaries for such cheap lands. All these tracts have increased in value to five, ten, one hundredfold, and more.

It is probably true that men never take the first step, looking to their own defense and social improvement, until they begin to suffer. Groups of people do not think. Thinking is an individual matter, and progress comes from individual effort rather than group action. It should be remembered that the surveyor has rendered an extremely valuable service to his country. He has been a lonely character; has undergone privation; has received but little in the way of compensation or praise; has suffered many physical hardships; and has seldom been understood by the few who knew him personally. If there are any glaring defects in the surveys, they are not to be charged to the surveyor. The service that he has performed represents one of the many gifts to an ungrateful society. He is very much like the pioneer class to which he frequently belonged. He performed the fundamental work which supports practically all development. His lines and angles cannot be seen, and the careless public, which enjoys all the ensuing benefits, never extends its thanks nor makes any record of the great debt it owes.

The wealth of the country lies in its lands. Only the skilled surveyor has the training and experience to locate boundaries properly. If he should claim to establish boundaries, he would find himself entangled with the law and all the machinery that the law has set up. Volumes have been written by men who consider themselves to be experts in the law of surveying. A test of a theory is its application; a test of a law or Court decision relating to surveying can only be made in the field, and the surveyor knows much more about the application of laws pertaining to surveying than the lawyer knows about surveying.

The surveyor and surveying itself have been injured by the attitude of many engineers who believe, or argue, that it is a rather low-grade profession, if a profession at all, and that but little training is necessary for one who plans to enter that field. This attitude has made it impossible for many engineers to appreciate the necessity for accurate surveys when accuracy is essential. They have an idea that surveying is performed in some miraculous way and that, as far as they are concerned, they need pay no attention to it. An analysis of the engineering work connected with any large structure will disclose that a considerable part of the skilled scientific work is surveying.

Instead of encouraging men to continue in surveying work, many engineers consider it a field for the novice and the beginner. This is injurious to the

public, to the Engineering Profession, and to the individual who is obliged to undergo the depressing experience. The surveyor and his art have also suffered because scientific branches of the National and State Governments have obtained control of surveying activities and have placed them under the supervision of men who, by training and experience, are not qualified to appreciate or properly direct the work entrusted to their care.

Although errors were made in public land surveys, the monuments set in connection therewith must be respected until some other system of control is provided. Private land surveys are connected in some way with these official monuments, many of which cannot now be found. Original surveys were not made carefully enough in many cases for the lines to be retraced and the position of monuments to be determined by following the notes of the public land survey. Many competent surveyors find themselves faced with insoluble problems when they attempt to rely wholly on the official notes. Some of these cases lead to litigation.

In many such instances, considerable damage is done because those who have power to render decisions are unable to appreciate conditions on the ground. Often the surveyor is able to induce all interested freeholders to agree as to the positions of monuments in question. He thus renders a great service. Again, the surveyor may find it more profitable to give testimony in the Court than to advise freeholders to pursue a peaceful course. If he makes a good appearance as a witness he may become detrimental rather than helpful to the community he should serve.

If freeholders realized how much money is spent by the great business interests to avoid litigation, they would have more respect for the advice of the surveyor when he recommends a settlement outside the Court. The retracing of old surveys is an art as well as a science and freeholders should understand that the competent surveyor, acquainted with local conditions, is in a position to render the best technical service and give the soundest advice as to procedure.

In the Eastern United States, conditions are more complicated, because lands were not surveyed under a uniform system, but have been in the hands of freeholders for a long time, and the Courts have decided many cases arising from boundary disputes. Some Courts in the East are inclined to minimize the importance of monuments and hold that the best description of land is afforded when the bounding freeholds are recited. This is undoubtedly a natural tendency of the legal mind. The layman can readily understand a map which shows the lands which border the tract in question. It is difficult for the legal mind to understand mathematical lines and the points of intersection where monuments should exist. The lawyer seldom appreciates the helpless position of the surveyor when he advocates or gives descriptions which do not refer to boundary lines and monuments.

While lands were cheap, it was impossible for freeholders to engage in expensive quarrels relative to boundary lines. As lands increase in value, boundary lines should be determined more accurately. The qualifications for this work should improve as money becomes available to pay for better service.

The training now offered by engineering schools is often not sufficiently thorough to prepare young men for this kind of work.*

Land surveys are of more general importance to the American people than many other branches of applied science combined. Without the ability to determine and record the boundaries of private lands, modern society would lose a part of its foundation. The importance of this kind of work is steadily growing. Teachers of engineering should stimulate practice and see to it that the qualifications for those proposing to engage therein are strictly prescribed. The survey of a boundary is public business. The registration boards of the various States, having power to define public work of an engineering character, should announce in definite terms that all surveys relating to boundaries are public work. No survey of private lands should be made except where a complete closure is possible. There should be National standards limiting the maximum errors.

The time may not be far distant when all monuments marking boundary lines between freeholders may be connected to a system of control. Then the monuments set by the public land surveyor may be ignored. The public looks to the engineer for service in connection with private land surveys, and he should not attempt to avoid this responsibility. It would be much better to improve the practice and to make the field attractive so that young men might have an opportunity for service therein. An attitude of carelessness toward surveying spreads to other branches of engineering. They, too, are becoming unattractive, largely because engineers have not stood together as a unit demanding high qualifications for admission to practice and just compensation for service rendered.

This careless attitude has led to handbook instruction in colleges and rule-of-thumb methods in practice. Freshmen are shown how to use transits and levels and other surveying equipment. Many educators defend this method as giving the young engineer some contact with engineering practice early in the course. This tendency is not prevalent in other professional instruction. A physician necessarily employs much scientific equipment, but medical schools do not try to interest freshmen and sophomores by teaching them how to use the instruments of the surgeon. Teachers of medicine have concluded that those who propose to become physicians should know something about anatomy, biology, and bacteriology, before they begin to use the practical equipment of the profession.

Private lands are continually being broken up and the volume of detailed surveying constantly increases. Many homesteads of 160 acres are now incorporated within the municipal limits of cities and towns. The surveyor has encountered many problems as this sub-division work has progressed. He should have some knowledge of other systems of municipal development and should be a stimulating force in his community rather than a hired man. His education should develop a respect for science. He should know something about the history of land tenure and enough about the law to enable him to keep his clients out of Court. He should understand the best examples of municipal

* A helpful manual on land surveying, issued by Mr. F. Hodgman, of Climax, Mich., in 1891, is the best that has been published.

development in the world and have the imagination to be able to suggest improvements. If the engineers of the country were represented by a large group of surveyors who could stimulate interest in matters of local concern, they would not be so generally misunderstood and under-appreciated. As long as certain branches insist on adhering to the Engineering Profession and are unable, because of a lack of training or proper organization, to render adequate service and make a definite impression on the public mind, the entire Engineering Profession must suffer.

Political Boundaries.—Surveys for political boundaries are now conducted by engineers qualified both by training and experience. As early as 1763, Mason and Dixon came from England to establish the line which still bears their names. This interest in, and recognition of, special ability furnished a valuable precedent. Many political boundary lines follow parallels of latitude and meridians. These astronomical lines cannot be located on the ground by amateurs in science. Where errors have been made in marking political boundaries, re-surveys by more competent men or by using better equipment, or both, have furnished results which have definitely settled boundary disputes.

County and township boundaries often follow lines delineated by public land surveys. As nearly all cities are expanding their boundaries, however, their limiting lines are constantly shifting. Surveys made at the time settlements were first established were generally poor and cheap. As villages, towns, and cities developed on ground formerly occupied by pioneer settlers, the volume and quality of surveying have steadily increased. Old monuments, which formerly marked the boundaries of lands owned by settlers, are frequently located in inconvenient and often inaccessible places, as under expensive buildings. Cities gradually lose the points of control which designated and protected the boundaries of freeholders and extend their triangulation control to provide carefully located monuments, placed where they may be protected and are accessible.

Ultimately, all municipalities will be obliged to accept this kind of control. Triangulation systems will extend from town to town and all rural, as well as urban, development will be tied together in a single, great network. With the rapid extension of all lines of communication the necessity for accurate control is evident. As cities have furnished some of the best examples in government, so they will stimulate interest in boundaries of every description.

Riparian Boundaries.—Men have never given much thought to natural resources which cannot be exploited or used in an exclusive way. The older civilizations have long since outlined and accepted wise principles relating to water resources. While the United States has received much benefit by the adoption of the English common law, it must be remembered that this law was developed in countries where the necessity for a public administration of water was not a matter of serious concern to the people at large. The common law came into being under feudal government. It has gradually changed and improved under the influence of individuals possessing ability, clothed with power, and interested in reform.

A combination of power, ability, and interest in improvement is a rare thing. Frederick the Great, of Prussia, was an outstanding example. This generation probably owes more to him and to a few similar characters than it commonly admits. The lawyers and Courts of his time were fettered by traditional law, many phases of which did not presume to lead to just decisions between man and man. In Carlyle's "Frederick the Great", is an account of one case relating to waters (known as the "Arnold Mill" case), in which that great ruler and administrator took part.

During the early years of the common law, waters were considered either as Crown property or as belonging to no one in particular. Under feudal government, the Crown and the aristocracy owned the lands, and the laws relating to real property and to waters were in the nature of agreements between proprietors. Conflicts relating to waters did not frequently arise because life was simple and the uses of water were few. The common law of England was carried across the Atlantic, and, when the American Colonies were converted into States, the common law was made available where statute law did not exist. Climatic conditions along the Atlantic seaboard were not much different from those of England and Northern Europe, where the common law had its origin.

Little study has been given water resources where the rainfall is ample and where streams are more important as drainage channels than as sources of water supply. Most of the rules relating to the waters of the Eastern States are an outgrowth of common law doctrines. Few people have given the subject sufficient time to appreciate many of the weaknesses of the common law rules or to make constructive suggestions looking to a system of administration which would protect all public and private interests and stimulate development.

There seems to be no good reason why feudal doctrines relating to water, accepted by a few landlords several centuries ago, should still meet the needs and interests of modern society. Unfortunately for this country, the only element that has had the final power of decision in cases of this kind is trained in the common law. Due to constitutional provisions and many precedents, the Courts are greatly handicapped. Should a judge understand the Roman law of waters or the principles carried into effect by Spain, India, and other countries representing older civilizations, he might be unable to apply them. The arid countries of the world have made the most progress in the administration of waters. When people begin to suffer, they begin to investigate the cause of their afflictions. The common law of England, relating to waters, cannot be applied very long in arid countries before its defects become apparent.

Ignorance as to the best principle which should be applied to waters immediately affects the prevailing attitude toward lands which underlie or border waters. Statute law was presumed to replace common law as rapidly as legislative bodies recognized opportunities for improvement. In the humid part of the United States it may be said that statute law relating to waters does not exist. The Western States have made progress, but, due largely to the influence of the common law, much confusion exists there relative to the administration of lands which form the beds and banks of streams and lakes.

No one can follow the history of any particular development in the United States without becoming something of an optimist. Public lands were a source of private speculation for fifty or sixty years. Finally, public interest and the protection of settlers won the battle. Forest wealth was largely destroyed before the people at large began to think about the value of a timber supply. During the last half century public sentiment has changed rapidly with respect to other resources and to the interests of the generations that are to follow.

Even under the common law the body or *corpus* of water is public. Any rule in conflict with this principle would fail in practical application, because it would be opposed to natural law. An understanding of waters and of the responsibility of society in their proper administration is necessary in any competent discussion of riparian boundaries. It should be apparent to the lay mind that if the timber supply is inadequate people will suffer for a generation or two. Trees would then be planted, and, in time, normal conditions would return.

Lumber is a vegetable product which Man can control; the supply of water is fixed by Nature. Every community must depend on local streams and lakes. Mistakes in judgment relative to the administration of water resources always injure many who are unable to protect themselves. If the people of the country had made an ample study of the many uses of water, the necessity for a public administration of water resources would have quickly led to some rational and definite settlement of riparian boundaries.

Statute law is silent regarding the claims of freeholders to banks and beds of streams and lakes. The common law reflecting the interest of feudal lords, gives color to these private claims. Property is private when it can be beneficially and exclusively used by the persons claiming it. Banks and beds of streams and lakes cannot be privately used, except in a very minor way and then often at the sacrifice of public interest. The maximum interest riparian claimants may represent is speculative rather than operative. The waters themselves, under all laws, are public. These waters must have basins (called "lakes") for storage, and channels (called "rivers") for the surplus water to escape to the sea.

Under the Federal Constitution, Congress has authority over navigable waters. There is no final definition of "navigable waters". If all uses of water were listed in the order of their vital importance to Man, "navigable" uses would be found well toward the bottom, in so far as rivers and lakes are concerned.

A public administration of water resources does not mean hostility to private development; rather that private development will be safeguarded and permitted where private uses are not in conflict with vital, public interests. Riparian boundaries may be established on land. They are unsatisfactory when they are under water, even for a brief time each year. If those who make decisions, and those who write textbooks, relating to riparian boundaries would take part in field work they now discuss, or attempt to define, they would learn how much easier it is to sit in a comfortable office and deal in generalities than

it is to go into the field and face conditions with which Man cannot compromise.

Instructions to public land surveyors directed them to meander the banks of all streams having a right-angle width of 3 chains, or more. The meander line, under these instructions, runs at mean high-water elevation where monuments were placed. It seems that the regulations governing this work referred to mean high-water elevation in order to afford a general guide to the surveyor. It was his duty to exercise his best judgment on the ground and to locate a definite boundary for the meander survey. There is some evidence that this was to be accepted as the boundary line between private and public property for all time. Since neither the law nor the regulations of administrative officers stipulate that these surveys relate to boundaries, the Courts have not frequently construed them as such. In an effort to protect what the Courts often regard as private property rights, the importance of the meander surveys made under the authority of law and under regulations issued by competent public authorities may be wholly or partly ignored. Entrymen who obtained patents from the Government to lands bordering on a meandered lake or stream thereby secured title to no lands beyond the meander survey. Private invasions of banks and beds of streams and lakes have taken place under the decision of Courts (thus reflecting the influence of the common law doctrine) rather than by the operation of Federal law and regulation of administrative officers.

The Federal Government had another reason for meandering wide streams and lakes. Freeholders could not afford to lose the use of submerged lands, so the Government made plans for their exclusion wherever the area might be sufficient seriously to handicap the prospective settler. River banks and lake shores which are flooded each year for longer or shorter periods, are necessary from the public standpoint, for if they did not exist serious floods would be more frequent. Considering rivers and lakes as parts of natural drainage systems, they should be publicly administered if individuals and the public are to be afforded uniform protection. This administration should be authorized to establish boundary lines between private and public property along lakes and streams where meander lines do not already exist. Laws of this kind are not likely to develop if initiative is delegated to those who have selfish interests, as "riparian proprietors", or to those who view the entire field in the light of the common law. The boundaries between private and public property should be determined definitely and all controversy between private and public rights and interest beyond these lines should cease.

Wherever individuals may use the banks or beds of streams and lakes without detriment to public interest, a permit may be issued therefor by public authority. This would be in the nature of a license subject to modification or cancellation whenever it may be shown, at a public hearing, that the use conflicts with public interest. This kind of public activity should be considered as administrative work, as decisions must rest on technical data and scientific facts which should be judged initially by a properly qualified public officer. As the critical data must be secured in the field, so the decision, to a large extent, should be rendered on the ground where all persons interested may understand

the problems involved and appreciate the purpose and scope of the rulings that are made. It is as difficult to view a drainage basin or a considerable part of a river system from a court room as it is to bring farming operations under a roof. If drainage is a public responsibility, there can be no compromise between private and public interest in the lands forming the beds and banks of streams and lakes. Extending the study into other uses of water, the public is found to be involved in all, and in many of them in an exclusive way.

Current thought relating to this subject has a definite trend toward public responsibility in so-called riparian lands. Modern Court decisions are rapidly running in the same direction. The engineer and surveyor who, almost alone, are able to interpret scientific data relating to waters, should be in a position to give unbiased advice to those who are trying to discard the traditional doctrines relating to riparian boundaries.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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TOPOGRAPHIC SURVEYS

PROGRESS REPORT OF THE COMMITTEE OF THE SURVEYING AND MAPPING DIVISION*

By W. N. Brown,† M. Am. Soc. C. E.

The day has passed when either engineer or architect can afford to plan any project without complete topographic data as a basis. The use of topographic maps is ever on the increase. If those engaged in making the surveys are to meet the growing requirements successfully, the differing conceptions of topographic surveying must be unified and methods standardized.

To many, a topographic map is still only a picture to be used as a general guide. To others, it is an accurate representation to a fixed scale of the existing physical condition of the terrain, upon which detailed plans may be safely predicated. These plans may also be staked on the ground directly from the map. Naturally, the requirements of map accuracy and fidelity of detail are entirely different for the two differing conceptions.

The members of the Committee on Topographic Surveys feel that methods in general use by the profession have not kept pace with improvements developed by organizations making a specialty of such work. Most prominent among such special organizations should be mentioned the U. S. Geological Survey and the U. S. Coast and Geodetic Survey.

In order to obtain better contact with existing practice, a questionnaire was sent to 93 engineering colleges and 115 practicing engineers in various parts of the United States. Approximately 50% of these letters were answered. It is felt that these replies give a good cross-section of topographic practice in this country. They indicate a wide divergence of opinion as to the methods to be used and the results to be accomplished. The two principal methods advocated were the plane-table and transit stadia.

* Presented at the meeting of the Surveying and Mapping Division, Washington, D. C., April 26, 1928. This is a report of the Chairman of the Committee of the Surveying and Mapping Division on Topographic Surveys, which consists of Messrs. W. N. Brown, *Chairman*, W. M. Beaman, J. K. Finch, G. P. Hovenor, and G. H. Matthes. The report has not been acted on by the Executive Committee of the Division, or by the other members of the Committee.

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It is both surprising and significant that out of 49 replies from prominent practicing engineers, 20 should give diametrically opposite answers to 18 others equally prominent. As an example one man very emphatically states that the plane-table is the only method for taking topography and that the use of any other is antiquated, unprofessional, and out of the question. Another just as dogmatically believes that the plane-table is entirely impracticable, and that the transit method is the only one to be even considered.

It is quite evident that one-half the profession does not use the same language as the other half nor does it realize what the other half is doing. Engineers are following the Biblical injunction that one should not let the left hand know what the right hand is doing. This may be good policy as applied to charity, but not to topographic surveying. The efforts of this Committee will be toward giving information as to what both hands are doing. It will try to standardize topographic methods so that both hands will work together more efficiently and accomplish better results more economically.

Topographic surveying may be divided into two general classes: Topographic mapping as executed by the Government, and private or commercial surveys. Each class serving a different purpose has developed a distinct technique.

Topographic mapping by the two Government Bureaus mentioned usually has for its object the setting forth of general information covering large areas for the use of the public of this and future generations. Such maps must be uniform in the character of data shown, in their scales, and in the accuracy of results.

Private or commercial topographic surveys usually cover small areas at large scales. Many are made with the object of securing only information of a special character, usually to determine the feasibility of some particular project. In many instances, the engineer is justified in spending only so much of his client's money as may be necessary properly to obtain this special information. This often leads to apparent duplication of work for if the special information obtained by the first survey indicates that the project is feasible, then a re-survey of the area to obtain additional data becomes necessary. There is thus a marked difference in object between surveys of the two classes described, and this difference has been largely responsible for the development of different methods in use by the engineers engaged in these two classes of surveys.

The men engaged in Government surveys, having uniform work, have thoroughly standardized their methods. The area to be mapped is studied, the method of control planned, the scale and contour interval are selected, the standard of accuracy or limit of permissible error is designated, and each step is planned in proper sequence. Methods are designated for each operation to insure an accuracy for the results of that operation consistent with the requirements of the map as a whole. The results are efficiency and economy. These men are making the best maps that can be made for the money expended.

Topographic surveys of the second class, namely, commercial surveys, have not been standardized; in fact, conditions are chaotic. Usually, the data are

wanted in a hurry. The engineer in charge often is more interested in other phases of the work, and so gives little real thought to planning the survey. The field men do their bit toward making a misfit operation. There is too much hurry to "fool with" proper control, or control is considered too expensive. Lines are run and closure errors accumulate. Then lines are re-run to locate the trouble. Both time and money are thus wasted. When the desired data are finally obtained, the project hurries on into construction, where the uncontrolled errors become still more troublesome.

Another reason for lack of standardization in commercial surveys is that the requirements of each project differ so greatly. This emphasizes again that the language spoken by the men in each class is different and they do not understand one another. If this Committee can adapt some of the best practice from the standardized surveys to the needs of commercial surveys, it will have done a great service. The Committee's efforts are being concentrated along the following lines.

Control.—The conception of a topographic survey as an accurate representation of the physical features of the terrain capable of serving as a basis for a paper location and as furnishing the necessary data for staking out such location on the ground, presupposes a certain accuracy of results. Such accuracy cannot be secured reliably without some system of control. Strictly speaking, no topographic survey, nor, for that matter, any survey, should be made without some form of control. The Committee intends to draw up specifications for the form of control best suited to each type of topographic survey, emphasizing that control is not only a necessity for proper work but may be justified on the grounds of economy over the present practice which tends to dispense with it. Control, in the minds of many, is associated only with triangulation or other forms of precise work, which are too complicated and expensive for the ordinary job. There seems to be a prejudice against it as being an expensive "frill."

Control, as used herein, denotes all points located with a sufficient degree of accuracy to prevent the accumulation of errors in excess of a predetermined limit. It is the Committee's duty to convince engineers that control is the framework on which the remainder of a survey is hung; that it is just as important to any survey as either the foundation or the structure of a building.

Accuracy.—At the beginning of every topographic survey it is important that a fixed standard of accuracy be decided upon. This does not mean that every survey must have the same standard, but it does mean a definite standard conforming to the requirements of the particular case. There is a definite relation between the accuracy required and the map scale on which the data are to be platted.

There should then be issued written instructions covering the field operations necessary to produce accuracy within the prescribed limits. Otherwise, money will be wasted in the field, for there will be as many conceptions of what is to be done as there are field party chiefs. Money will be uselessly spent in doing some parts of the field work more accurately than necessary, while in others, operations will not be done with sufficient accuracy to maintain the standard set for the whole, thus vitiating the entire map.

The Committee will attempt to standardize these instructions for different limits of allowable error so that surveys will be uniformly made in all their component parts. There is great need also for standardizing map symbols and map scales, assuming a fixed relation between the scale and the accuracy desired.

Monumentation.—One of the most important things in connection with a topographic survey, is the proper referencing, or monumentation of a sufficient number of accurately located points to enable the easy projection on the ground of the subsequent paper plan. Yet, in many cases, topographic surveys are made, and no permanent monuments are set to mark the control points, with the result that six months later, there is no satisfactory way of referring accurately from the map to located points on the ground.

Aerial Photography.—This has been successfully used since about 1923 as an aid to topographic surveying, even developing into a means in itself of making such surveys. Under certain conditions it offers many advantages. Yet the colleges are not teaching it, nor is the Engineering Profession as a whole making use of it. The Committee proposes to give data on the best methods of utilizing this new aid to the topographer.

Each member of the Committee has taken a definite task in writing the specifications best suited for different types of work. These will cover:

(1) Government topographic surveys for scales of from 1:10 000 to 1:100 000.

(2) City topographic surveys for scales of 1 in. equals 100 ft. to 1 in. equals 400 ft.

(3) Private or commercial surveys for preliminary investigations, at scales of from 1 in. equals 200 ft. to 1 in. equals 2 000 ft.

(4) Commercial surveys for detailed planning purposes, using scales of from 1 in. equals 20 ft. to 1 in. equals 200 ft.

The first two are about completed and good progress has been made on the others. The Committee would appreciate any help from members either by discussion or by letter.

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LOCATION AND CONSTRUCTION SURVEYS

PROGRESS REPORT OF THE COMMITTEE OF THE SURVEYING AND MAPPING DIVISION*

By W. C. Taylor,† M. Am. Soc. C. E.

Among the old books in the Library at Union College is one entitled "The Compleat Surveyor", written by William Leybourne and bearing the date 1679. The dedication reads as follows:

"To the Right Worshipful Sir Thomas Player, Knight,
Chamberlain of the Honourable City of London.

"Sir,

"As the excellency of your mature judgment in Arts and Sciences hath over-perswaded me, so the wonted goodness of your natural disposition doth further invite me, to shelter these ensuing Tractates under the Tutelage of your Worship's Patronage, whereof Geometry is the subject; a matter of that woth and eminency, that Philo (the Jew) termed it the Princes and Mother of all other Sciences; receiving its originall Name from Measuring the Earth; for when Nilus by his Inundation drowned and confounded the Aegyption Banks, by help of this Geodeticall Art (after the Fall of the Waters) every man had his portion of Land restored to him again. And how much occasion we have had to make use hereof in the late London Ruines, your Worship cannot be insensible, and my self (being several times by the Cities's favour employed therein) can eminently witness. Now how necessary Geometry is, not only in Civill, but in Military Affairs also, I refer to your Worship's censure, whose love and affection to, as well as profound knowledge therein, is best able to determine. For by the assistance hereof, the Military Architect may fabricate his Fort; the Engineer spring his Mines, raise his Batteries, plant and mount his Cannon to best advantage, and greatest annoyance of his Enemies; the General or Major lay out his Ground for Quarters, encamp his Army, etc. And feeling there is so great an affinity between the Art Military and the Sciences Mathematicall, I (in confidence of your Worship's friendly

* Presented at the meeting of the Surveying and Mapping Division, Washington, D. C., April 26, 1928. This is a report of the Chairman of the Committee of the Surveying and Mapping Division on Location and Control Surveys which consists of Messrs. W. C. Taylor, *Chairman*, C. A. Garfield, Robert Hoppen, Jr., W. B. Landreth, and J. H. Myers. The report has not been acted on by the Executive Committee of the Division, or by the other members of the Committee.

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acceptance, and pardon of this my presumption) have adventured to commit the same to your Worship's favourable reception. And the rather, for that these few years since London's dreadful Conflagration, I have been employed in the Admeasurements of the Publick Works, belonging to this Honourable City, in which your Worship is so deservedly dignified both in Place and Person, and wherein you so admirably discharge the trust committed to you, to the satisfaction of all good citizens, (of which my self am one of the least). I shall not longer detain your Worship from your more important affairs, which you are continually busied with, but once more crave pardon for this my boldness; In the assurance whereof, as I shall always pray for the welfare of this City, so for the principal Governours, and particularly for your Worship, to whom be continuance of Health and encrease of Honour. And so I subscribe my self,

"Sir,
"Your most faithful and obedient Servant,
"W. LEYBOURNE."

It is interesting to note in this book 250 years old, that surveying in those early days was devoted chiefly to two purposes: First, Boundaries; and, second, Military Operations.

One hundred years later, in the United States, the demand for surveying was also along these same lines. The Revolutionary War brought out the need for good maps for military purposes. In 1777, Congress, in response to this need, authorized the appointment of a "Geographer and Surveyor of Roads"; and in 1813, the appointment of a Corp of Topographic Engineers, under the General Staff of the Army.

About the same time the desire to open up the great expanse of territory west of the Alleghanies, and across the Mississippi, brought about the authorization of the public land surveys, which resulted in the organization of the General Land Office in 1812.

With the beginning of the nation means of communication between the States and for the transportation of goods to and from the seaport towns led to the construction of turnpikes; also, roads were projected into the West. The most famous of these was Wilderness Road through the Cumberland Gap, laid out in 1775 by Daniel Boone. In speaking of it, one historian says:

"The location of the road, however, is a monument to the skill of Boone as a practical engineer and surveyor. It required a mind of far more than ordinary calibre to locate through more than two hundred miles of mountain wilderness a way of travel, which, for a hundred years, has remained practically unchanged."

The beginning of location surveying was thus in connection with these early turnpikes and roads. For a period of fifty years, or more, a great many turnpikes were laid out, some quite extensive, such as the great National Pike, originally running from the Potomac to the Ohio River and, later, extended into Indiana and Illinois. Another famous one was the Lancaster Turnpike in Pennsylvania, chartered in 1792, from Philadelphia to Lancaster, 62 miles. This was a regular graded and stoned turnpike. It proved so beneficial that it was later extended to Pittsburgh, 303 miles distant, with "ascents not exceeding 3¾ degrees", or about 6½ per cent. Hundreds of miles of turnpikes were constructed in the period from 1800 to 1825.

The characteristic of most of these turnpikes was their straight alignment for several miles. The grades were in some cases rather steep; the maximum on the National Turnpike was 8¾ per cent.

Early in the beginning of the nation, in fact even before the Revolutionary War, there was a demand for the improvement of the rivers for navigation. Most of these attempts were not satisfactory, and there was considerable agitation for the building of artificial canals.

One of the first of these was the Potomac Canal sponsored by General Washington. It did not materialize until some years after, however, when the company was absorbed by the Chesapeake and Ohio Canal Company. Several other canals were projected and routes surveyed. Perhaps the most important was the Middlesex Canal in Massachusetts. For all these canals, European engineers were engaged, although most of the surveying was done by native Americans.

It remained for the Erie Canal to create the Profession of Engineering in America. Surveys for it were authorized in 1808. Some surveys and short canals had been made before this time, for from very early beginnings of the country there had been a desire to find a route from the Hudson River to the Great Lakes. This step, however, marked the beginning of real progress.

The first difficulty was to find a man who could conduct such a survey and plan such a canal. The man who had been active and had the ability was an Englishman, William Weston. He had recently returned to England, and refused the offer. It was necessary, therefore, to develop native talent.

In those days, judges and lawyers were surveyors, for a knowledge of surveying was useful in questions of deeds, property lines, etc. From this group,

"James Geddes, a judge and surveyor of Onondaga County, was chosen for this first survey in 1808. He surveyed from Oneida Lake to Lake Ontario where the Salmon Creek enters it; another line down the Oswego River to the lake; a line from Lewiston to the navigable waters of the Niagara River above the Falls and then from Buffalo east to the tributaries of the Seneca River. In this work he followed the best route that exists for a canal; and the whole was accomplished for the sum of $673."*

Geddes proved himself of remarkable ability and drew around him a group of men of unusual talents. One of these was Benjamin Wright, who has sometimes been called the "Father of American Engineering." He was also a Judge and Surveyor. He had had some experience in canals in Western New York and considerable skill with a leveling instrument.

In regard to this beginning, a son of James Geddes wrote:*

"As has been stated, all efforts to secure the services of the English engineer, Mr. Weston, having failed, the Commissioners were in great doubt as to the best course to pursue. Under these circumstances Mr. Geddes and Mr. Wright, having consulted with each other, appeared before the Board, and expressed their confidence in their ability to locate and construct the canals, but expressed a strong desire that the Commissioners should feel a like confidence, if they were to be entrusted with the responsibility. Most fortunately for the State, the Commissioners gave these engineers that confidence. But in so doing, they encountered the censures of the enemies of the canals, in

* Rept. of New York State Engr. and Surveyor on History of New York Canals, 1905.

and out of legislative halls. On the Assembly Floor, it was tauntingly asked, 'Who is this James Geddes, and who is this Benjamin Wright, that the Commissioners have trusted with this responsibility—what canals have they ever constructed? What great public works have they accomplished?' But, really, the Commissioners had no alternative—and now it is easy to see that the course adopted was much wiser than to have entrusted the canals to the keeping of any one man, as would have been the case had the efforts made to secure Mr. Weston been successful.

"To add still more to these difficulties in regard to the engineering, it was said in high places, by men who claimed much knowledge on such subjects, that no confidence could be placed in an ordinary engineer's spirit level for laying out long lines of canal, and that there was no possibility of running a line for the long levels, that was not liable to be erroneous to the whole depth of the canal. So much annoyance did these cavillers produce, that in the next year it was deemed expedient that in order to settle that matter a full test should be made."

This test consisted in a line of levels between Rome and Syracuse, N. Y., around Oneida Lake, forming a circuit of 100 miles. The closure was less than 1½ in. The critics were thus answered.

When the surveys on the Erie Canal were commenced, there was no model on this Continent that could be used as a guide, and few books on the subject. The country through which it ran was a complete wilderness, yet the work was done remarkably well and in a remarkably short time.

The report of the Canal Commissioners of New York State, of February 20, 1824, reads:

"None but those who had examined the line previous to the commencement of the work; who had seen the rude and undulating surface which was traversed, the rocks which were to be blasted, the irregular ledges filled with chasms and fissures which were to form the sides and basis of a water-tight canal; the spongy swamps, and gravel beds, and quicksands, which were to be made impervious to water, and, in short, the huge masses of rough materials, which, with immense labor, were to be reduced to symmetry and form, can duly appreciate the efforts which it has required to surmount these various obstacles."

Location and construction surveying in the United States had its foundations laid by this group of men gathered to construct the Erie Canal. For twenty-five years or more after this, there was an enthusiasm for canal building and, in 1840, there were 4 468 miles of canals in the United States.

The cities along the Atlantic Coast entered into keen competition for trade with the West. The problem facing them was to climb over the Alleghany Mountains, or, in the case of New England, to surmount the Berkshires. New York, however, had found a way by the Erie Canal and was destined to outrank her rival cities. Baltimore, Md., with its thriving sea-going trade was most alarmed and, in 1827, projected the Baltimore and Ohio Railroad, which was then a bold proposition.

The Army Engineers were called into service to make the preliminary surveys. As stated* by the late Mendes Cohen, Past-President, Am. Soc. C. E.,

"The duty assigned to these officers was performed with so much assiduity that they were enabled to present on the 5th of April, 1828, a report which

* *Transactions*, Am. Soc. C. E., Vol. XXVI (1892), p. 539.

covered a general examination of the whole intervening region between the Patapsco and Ohio rivers, and with so intelligent a discussion of many of the alternative lines that it is a matter of no little wonder how so much could be accomplished in so short a time through so rough a wilderness as the Alleghanies must have been in that day. The study of the route seems to have been based on the theory that the gradient of the road, to be worked probably by horses, possibly by locomotives, must be moderate, not exceeding 30 feet per mile, as nearly uniform as possible, never undulating, that a very liberal amount of lateral curvature was admissible in furtherance of this, and for the avoidance of heavy cuts and fills; and that where summits were to be crossed and great differences of level to be overcome, recourse should be had to inclined planes, to be operated preferably by steam or water power."

In 1830, the Directors abolished the Board of Engineers and placed the entire work in the hands of Jonathan Knight, as Chief Engineer. He had been in the employ of the Board of Engineers for about two years. Previous to this railroad work, he had been engaged on the National Pike.

The work of construction was slow, with a great many obstacles in the way. Relative to this, Mr. Cohen states:*

"During these years of obstruction, authority had been obtained to construct a branch road to Washington, and in 1831 Mr. B. H. Latrobe was appointed an assistant and assigned to the reconnaissance. This was followed up without delay by a more definite and careful examination. Mr. Knight takes a most comprehensive view of what he says must become a great national highway. He urges that no grades exceeding 20 feet per mile, nor any curve of less radius than 1 500 feet, or in extreme cases, 1 000 feet, be tolerated if avoidable at any reasonable expense, so that light locomotives may make the run regularly in two hours and indicates the possibility of a 'message being made to pass the whole distance from Washington to Baltimore in one hour.' The surveys were continued with great minuteness so that, as the President expresses it, 'the route finally adopted should leave no better one available to a rival corporation.'

"In July, 1833, Mr. Knight submits his report and analyses of twelve alternative lines with a degree of elaboration and care that I venture to say has rarely been equaled. The line recommended by him was at once placed under contract, and under Mr. B. H. Latrobe, as the engineer in charge, was completed and opened August 25th, 1835. At its point of divergence from the main stem 7 miles from Baltimore it crosses the Patapsco by a masonry viaduct of eight arches, each of 58-foot chord, some 70 feet in height above the water and of a total length of 707 feet. This was designed by Mr. Latrobe and stands to-day a monument to his taste and professional skill."

The Baltimore and Ohio Railroad did for the railroads what the Erie Canal did for the canals. Railroad location and construction went forward with great enthusiasm. In 50 years there were 96 500 miles in operation, and 190 000 miles had been constructed by 1900. In one year (1887), 12 982 miles were built. Railroad construction in the United States was the greatest engineering work of the Nineteenth Century.

In 1853-54 the Government authorized the surveys for the transcontinental lines from the Mississippi River to the Pacific Coast. The result of these surveys included reports on five possible routes† contained in thirteen large

* *Transactions*, Am. Soc. C. E., Vol. XXVI (1892), p. 549.

† "Reports of Exploration and Surveys to Ascertain the Most Practical and Economical Route for a Railroad from the Mississippi River to the Pacific Ocean."

quarto volumes. Each of the five routes was later used by transcontinental railroads. They were doubtless the most extensive location surveys ever made in this country.

The railroads occupied the attention of engineers to a great extent, yet along with them came the need of structures, and the growing cities had many demands within themselves. There were tunnels, subways, buildings, bridges, water supplies, and sanitary problems. For these, surveying played a much less prominent part than for railroads and canals and, consequently, engineers placed their emphasis less and less on surveying.

The problems changed. The scientific man supplanted the practical engineer. Surveying did not occupy the large part of the work of the engineer which it had been doing. Yet with all these engineering undertakings, the surveyor has his important place. The scope of his work has broadened. New demands have arisen.

Speaking in 1869, the late John B. Jervis, Hon. M. Am. Soc. C. E., said:*

"Fifty years ago, it was generally considered that the land surveyor had the basis of the engineer's art. To run lines and levels and map the same with topography of country, was held in high estimation as an art. At that time little was known of designing plans, and maturing the numerous details of works in engineering. Little had then been done in this country, and, of course, very few specimens were offered for study."

It is interesting thus to note that pioneer engineers developed from location surveyors. These pioneering days are over, and the question arises as to the place of the surveyor to-day.

As one looks over the country and notes the engineering construction in progress, the large water supplies, the enormous highway programs, the hydro-electric plants, the port developments, the subaqueous tunnels, the long-span bridges, the city planning, and many others in all parts of the country—projects of great magnitude—it would be difficult to imagine how these could be carried on without the aid of the surveyor. His requirements include an ever-increasing knowledge of all types of surveys and methods of precision. At times, this may mean precise triangulation control, accurate boundary location, or complete topographic delineation. His lines may be on the surface, under the surface, or through the water. He must keep in step with the advancing movements of engineering.

It is with this broadened scope in mind, that this Committee has prepared the following tentative outline of work. This outline is subject to change as the work of the Committee progresses.

Outline of Work.—

I.—Location and Construction Surveys:

1.—Nomenclature and Definitions.
2.—General Specifications.
3.—Classification of Surveys:

A.—Route Surveys:

(1) Canals.
(2) Drainage.
(3) Highways.

* *Transactions*, Am. Soc. C. E., Vol. I (1872), p. 144.

(4) Irrigation.
(5) Mines.
(6) Railroads.
(7) Sewers, Aqueducts, and Pipe Lines.
(8) Subways.
(9) Subaqueous Lines.
(10) Tramways.
(11) Transmission Lines.
(12) Tunnels.

B.—Development of Area Surveys:

(1) Airplane Landing Fields.
(2) Bridges.
(3) Buildings.
(4) Dams.
(5) Elevators.
(6) Flood Control.
(7) Grade-Crossing Eliminations.
(8) Industrial Plants.
(9) Institutions, Public and Private.
(10) Machinery Foundations.
(11) Municipal Plants.
(12) Park Systems.
(13) Port Development.
(14) Power Plants.
(15) River Bank and Shore Protection.
(16) Transportation Terminals and Yards.
(17) Wharves and Piers.

C.—Quantity Surveys:

(1) Dredging.
(2) Excavations.
(3) Earthwork.

The plan of the Committee is to bring together from all available sources, the recognized best practice along all these lines. From these data, manuals may be prepared for the Society. It is hoped thereby, that engineers who are skilled in one branch may be able to become proficient in another branch of the work.

The work of the Committee is much increased because there is so little literature available. Engineers in the past have written quite full reports and papers on many phases of their projects, but it is rare to find a detailed description on the survey or method of location. Apparently, the call for pioneering effort has come again to the location surveyor and in that spirit the Committee is pressing forward to its task.

As outlined, thirty-two subjects are to be treated. Although some of these will doubtless overlap, at best, the studies will take quite a period of time.

The Committee seeks the co-operation of all members of the Society in its efforts to bring this work to fruition.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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CONTROL SURVEYS

PROGRESS REPORT OF THE COMMITTEE OF THE SURVEYING AND MAPPING DIVISION*

By C. V. Hodgson,† M. Am. Soc. C. E.

The progress made to date by this Committee consists practically of a single report on Horizontal Control published recently.‡ The place this report occupies in the development of the subject of Control Surveys is shown by the outline which follows, and which corresponds closely to the schedule of reports which the Committee expects to prepare.

The Executive Committee of the Division has divided the subject of Surveying into four general classes, each covered by a main committee of the Division as follows:

General Classification of Surveys

I.—Location and Construction Surveys.
II.—Boundary Surveys.
III.—Topographic and Hydrographic Surveys.
IV.—Control Surveys.

IV.—Control Surveys.

Horizontal Control:

(A) Classification, definitions and general specifications.
(B) First Order:
  (*a*) Detailed specifications.
  (*b*) Instructions for field operations.
  (*c*) Instructions for office operations.
  (*d*) Forms for tabulation of final data.

Vertical Control:

(A) Classification, definitions and general specifications.
(B) First Order:
  (*a*) Detailed specifications.
  (*b*) Instructions for field operations.
  (*c*) Instructions for office operations.
  (*d*) Forms for tabulation of final data.

* Presented at the meeting of the Surveying and Mapping Division, Washington, D. C., April 26, 1928. This is a report of the Chairman of the Committee of the Surveying and Mapping Division on Control Surveys, which consists of Messrs. C. V. Hodgson, *Chairman*, E. M. Douglas, George L. Hosmer, A. B. Pierce, and R. H. Randall. The report has not been acted on by the Executive Committee of the Division or by the other members of the Committee.

† Asst. Chf., Div. of Geodesy, U. S. Coast and Geodetic Survey, Washington, D. C.

‡ "Horizontal Control in Surveying"; Preliminary Report of the Committee of the Surveying and Mapping Division on Horizontal Control, by George L. Hosmer, M. Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2491.

(C) Second Order:
Detailed outline, as for first order.
(D) Third Order:
Detailed outline, as for first order.
(E) Fourth Order:
(*a*) Specifications.
(*b*) Field methods.
(*c*) Office treatment.
(*d*) Discussion of applications.

(C) Second Order:
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The report by Professor Hosmer already issued, covers Part (A) under "Horizontal Control." It will serve temporarily as a guide to the other committees of the Division until detailed reports can be brought in for each class of control. In preparing it Professor Hosmer was guided by the international standards which have been recognized for many years by the leading nations of the world and which were embodied in a report submitted to the International Geodetic and Geophysical Union at its meeting at Prague, Czechoslovakia, in 1927, by its Committee on Triangulation.

The field to be covered by the Committee on Control Surveys is well defined. Complete and definite specifications for accuracy must be established for each grade of control, in order that any committee of the Division, or any engineer dealing with surveys, may refer to the specifications for any class of control and know that they will have a definite and well understood meaning. A manual of field and office practice will be prepared for each class of control, to fill the gap between the textbook on surveying and the fragmentary and uncoordinated instructions now available to the engineer in various books on field methods. It will be left to the other committees of the Division to specify the class of control for each kind of survey and the frequency of distribution of the control points. It will be the function of the Control Committee, however, to co-operate with the other committees whenever necessary, in the preparation of reports on the various kinds of surveys, and to stimulate the publication of papers on special problems relating to control surveys.

The distinction between public and private surveys depends on whether the initial purpose of the control is for Federal, State, county, and city surveys, or for private engineering projects. The more closely one investigates the situation, however, the smaller the difference appears, for a control point, once properly located and marked, is available for all purposes. The only fact an engineer wishes to know is, how accurately was it located? Private engineers all over the country are using the triangulation stations and level bench-marks established by the public surveys and, on the other hand, the public survey organizations use private survey data whenever available. This Division should be able, by its classification of control surveys, and by its specifications for the adequate marking of control points, to do much to make control surveys, by whatever agency established, of far greater value to engineers in general.

To a great extent so-called public and private control surveys supplement each other. Consider, first, the method of establishing a scheme of National

horizontal control, the purpose of which is to furnish a framework to support all detailed surveys and to give them proper latitudes and longitudes. Two preliminary steps are necessary: First, to decide on the shape and size of the spheroid which will most closely correspond to the shape of the portion of the earth's surface to be controlled; and, second, to adopt a latitude and longitude for a datum point. The first step was accomplished for the United States about 1880 by the adoption by the Coast and Geodetic Survey of the lengths of the earth's axes as determined by Clarke for his spheroid of 1866. The second step was taken about 1900 when the U. S. Coast and Geodetic Survey assigned to a triangulation station in Kansas a definite latitude and longitude and established the United States Standard Datum. Later, Canada and Mexico adopted the same datum, and it is now called the North American Standard Datum.

The first-order triangulation available at the time of the adoption of the U. S. Standard Datum is shown in Fig. 1. It consisted of little more than a belt along each coast and a single bar across the country. After the latitude and longitude were adopted for the datum point the geographic positions were re-computed on the new datum for all points in the existing scheme. As new arcs were added, the old points were held fixed and the new ones computed on the old base.

A triangulation system is an engineering structure and must be treated as such. The system as it existed in 1900, at the time of the adoption of the datum, had exactly the status of a beam supported and held fixed at the datum point near its middle and without adequate support at the ends. It provided poor support for a National survey structure, but it was the best obtainable at that time.

The National scheme of triangulation as it existed in 1927 is shown by the solid black belts in Fig. 2. The portion of the system west of the 98th Meridian looks like a vertical section of a framed structure. It was built up, however, a piece at a time, and did not fit exactly at the junction points prior to the mathematical adjustment of each belt of triangulation. Sometimes the process of adjustment forced errors into the triangulation which were much larger than the errors of observation. In other words, since the main beam of triangulation along the 39th Parallel was not exactly in its correct position, and since each member of the structure joined to the main beam was not exactly correct in length and direction, the whole structure needed truing up. Looking into the future it was seen to be necessary to re-adjust the main scheme framework in order to provide an accuracy of geographical position which would be adequate for all the requirements of a National survey for the United States for centuries to come. A new and economical method of making this adjustment was devised and outlined in its general principles by William Bowie, M. Am. Soc. C. E., Chief of the Division of Geodesy, U. S. Coast and Geodetic Survey. Dr. O. S. Adams, of that organization, perfected the method and made a mathematical application of it to the re-adjustment of the triangulation of the western half of the United States. The sixteen circuits west of the 98th Meridian closed with an average error of about 1 part in 450 000 after the discrepancies in length between

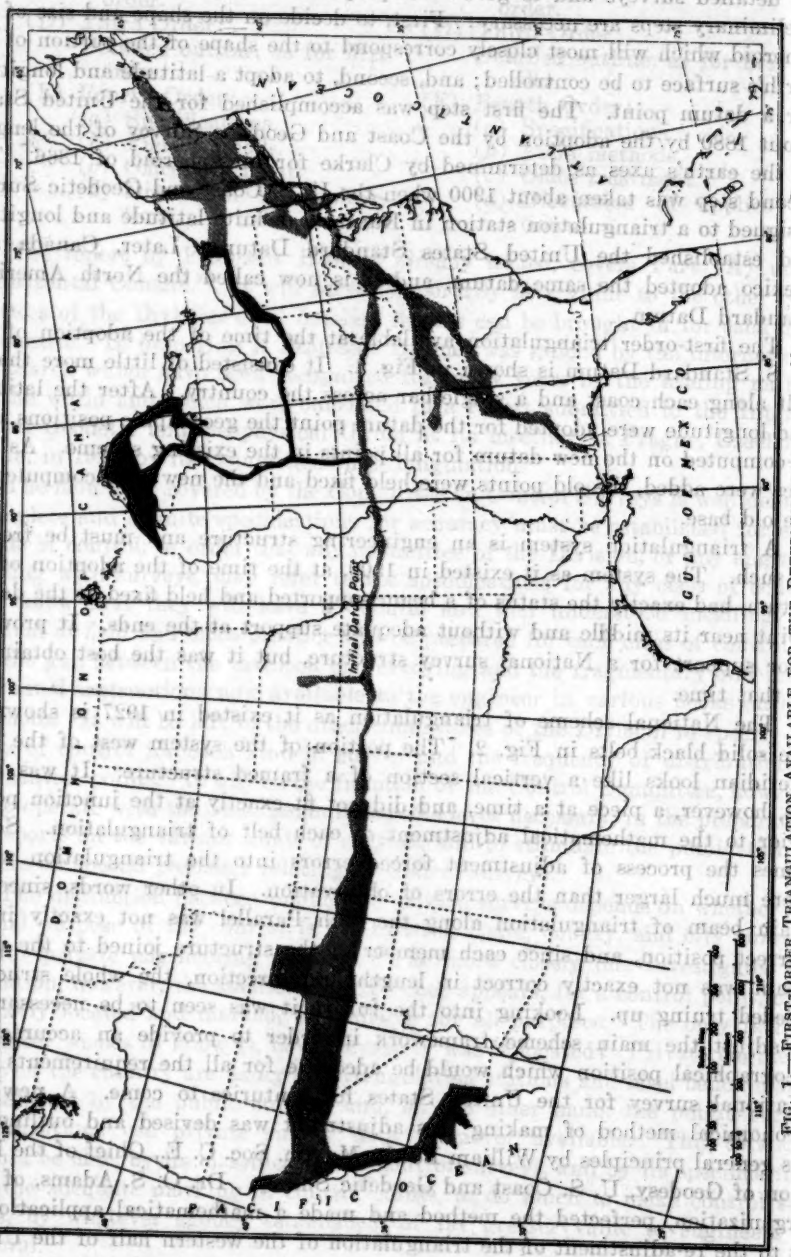


FIG. 1.—FIRST-ORDER TRIANGULATION AVAILABLE FOR THE DETERMINATION OF THE NORTH AMERICAN DATUM.

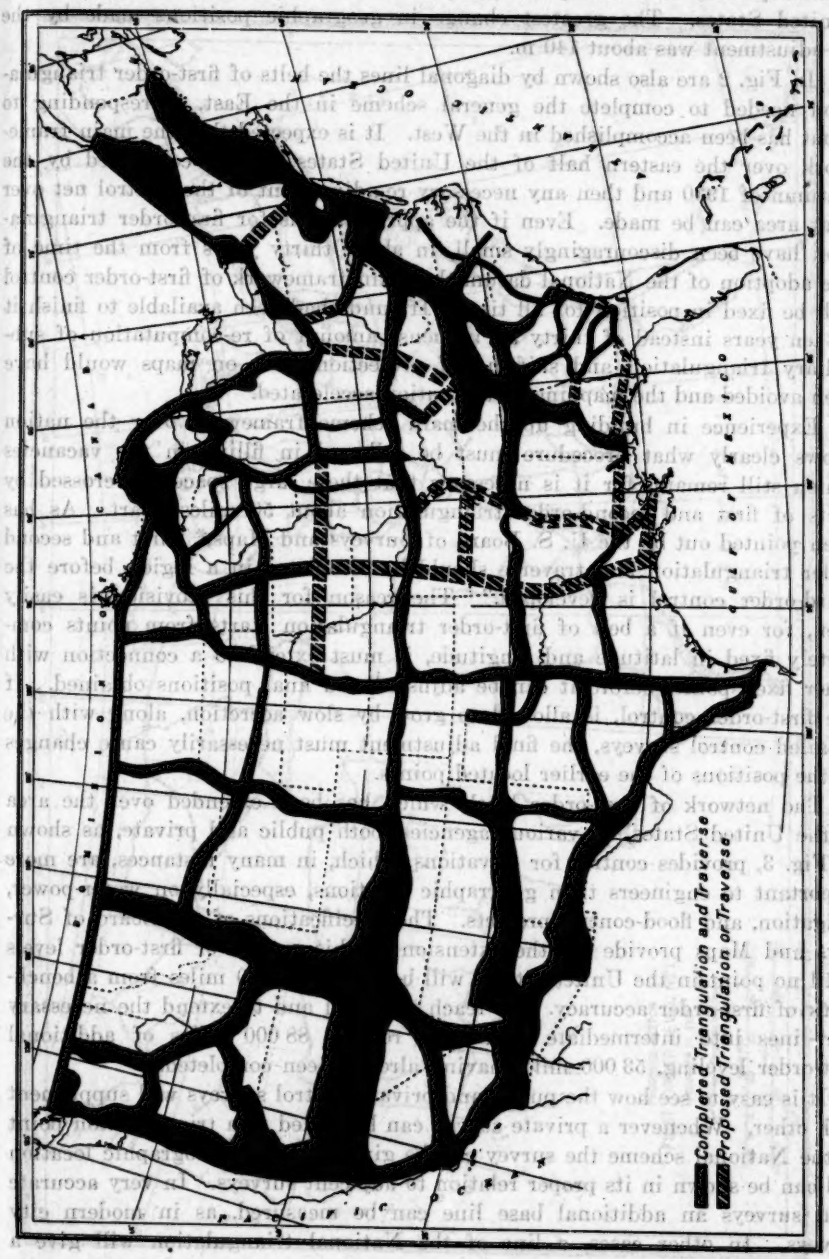


FIG. 2.—TRIANGULATION OF THE UNITED STATES, 1927.

bases and the discrepancies in azimuth between the fixed azimuth stations had been adjusted, an accuracy adequate for even a continental area such as the United States. The greatest change in geographic positions made by the re-adjustment was about 140 m.

In Fig. 2 are also shown by diagonal lines the belts of first-order triangulation needed to complete the general scheme in the East, corresponding to what has been accomplished in the West. It is expected that the main framework over the eastern half of the United States will be completed by the autumn of 1930 and then any necessary re-adjustment of the control net over that area can be made. Even if the appropriations for first-order triangulation have been discouragingly small, in about thirty years from the time of the adoption of the National datum the main framework of first-order control will be fixed in position for all time. If funds had been available to finish it in ten years instead of thirty an immense amount of re-computation of subsidiary triangulation and shifting of projection lines on maps would have been avoided and the mapping of the nation accelerated.

Experience in building up the main scheme framework over the nation shows clearly what procedure must be followed in filling in the vacancies which still remain, for it is necessary that these large spaces be crossed by belts of first and second-order triangulation about 50 miles apart. As has been pointed out by the U. S. Board of Surveys and Maps,* "first and second order triangulation and traverse should be completed in a region before the third-order control is developed." The reason for this provision is easily seen, for even if a belt of first-order triangulation starts from points completely fixed in latitude and longitude, it must extend to a connection with other fixed points before it can be adjusted and final positions obtained. If the first-order control, is allowed to grow by slow accretion, along with the detailed control surveys, the final adjustment must necessarily cause changes in the positions of the earlier located points.

The network of first-order levels which has been extended over the area of the United States, by various agencies, both public and private, as shown in Fig. 3, provides control for elevations, which, in many instances, are more important to engineers than geographic positions, especially on water-power, irrigation, and flood-control projects. The specifications of the Board of Surveys and Maps provide for the extension of this system of first-order levels until no point in the United States will be more than 50 miles from a bench-mark of first-order accuracy. To reach this goal and to extend the necessary spur-lines into intermediate areas will require 88 000 miles of additional first-order leveling, 53 000 miles having already been completed.

It is easy to see how the public and private control surveys will supplement each other. Whenever a private survey can be joined to a triangulation point of the National scheme the survey can be given a correct geographic location and can be shown in its proper relation to adjacent surveys. In very accurate local surveys an additional base line can be measured, as in modern city surveys. In other cases, a line of the National triangulation will give a length and azimuth for the local survey as well as a geographic position. In

* Report on Federal First-Order Control Surveys adopted in 1926.

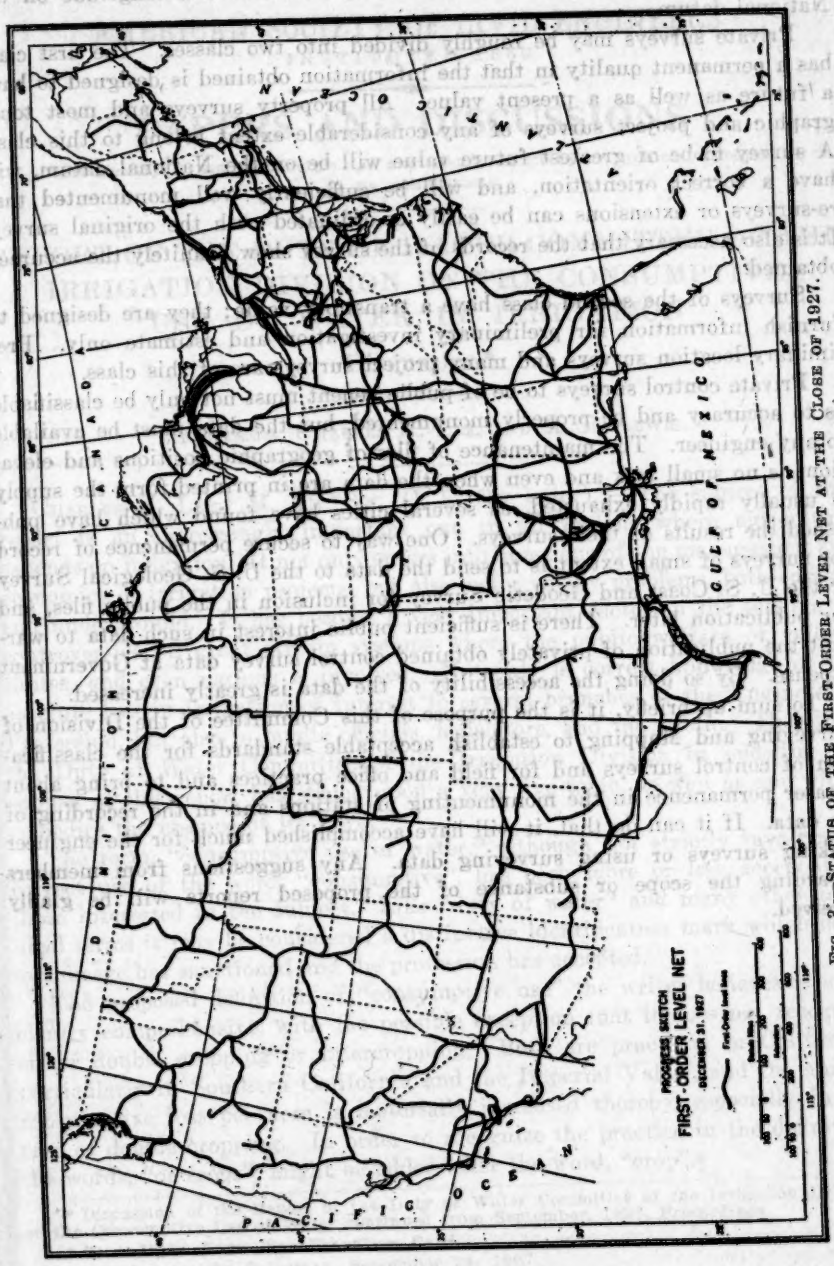


FIG. 3.—STATUS OF THE FIRST-ORDER LEVEL NET AT THE CLOSE OF 1927.

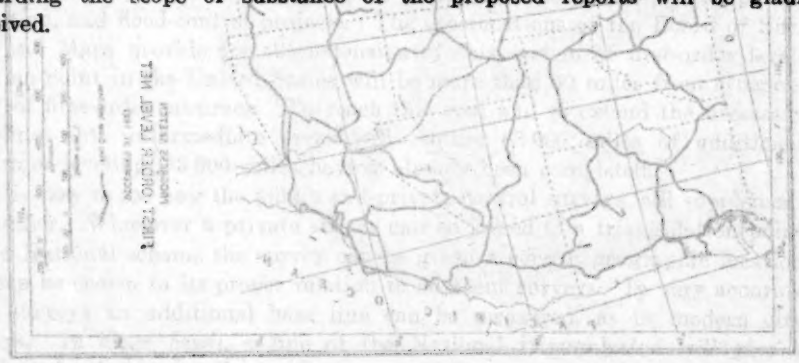
most private surveys the area covered is relatively small and plane co-ordinates should be used, the origin having a correct latitude and longitude on the National datum.

Private surveys may be roughly divided into two classes. The first class has a permanent quality in that the information obtained is designed to have a future as well as a present value. All property surveys and most topographic and project surveys of any considerable extent belong to this class. A survey to be of greatest future value will be on the National datum, will have a correct orientation, and will be sufficiently well monumented that re-surveys or extensions can be easily co-ordinated with the original survey. It is also necessary that the records of the survey show definitely the accuracy obtained.

Surveys of the second class have a transitory value; they are designed to furnish information for preliminary investigation and estimate only. Preliminary location surveys and many project surveys are of this class.

Private control surveys to be of public benefit must not only be classifiable as to accuracy and be properly monumented, but the data must be available to any engineer. The maintenance of files of geographic positions and elevations is no small task and even when the data are in printed form the supply is usually rapidly exhausted, as several cities have found which have published the results of their surveys. One way to secure permanence of record for surveys of small extent is to send the data to the U. S. Geological Survey or the U. S. Coast and Geodetic Survey for inclusion in the public files, and for publication later. There is sufficient public interest in such data to warrant the publication of privately obtained control survey data at Government expense. By so doing the accessibility of the data is greatly increased.

To sum up briefly, it is the purpose of this Committee of the Division of Surveying and Mapping to establish acceptable standards for the classification of control surveys and for field and office practices and to bring about greater permanence in the monumenting of stations and in the recording of the data. If it can do that, it will have accomplished much for the engineer making surveys or using surveying data. Any suggestions from members regarding the scope or substance of the proposed reports will be gladly received.



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REPORT OF THE DUTY OF WATER COMMITTEE OF THE IRRIGATION DIVISION ON THE CONSUMPTIVE USE OF WATER IN IRRIGATION

Discussion*

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CHARLES H. LEE,† M. AM. SOC. C. E. (by letter).‡—The subject of this report is an increasingly important one in all regions where agriculture depends on irrigation. It not only enters into questions of the management and economics of irrigation projects, but also water supply problems, both surface and underground. It has become a very important factor in the settling of controversies on great stream systems where the public welfare of valleys, States, and even nations is involved. Not only is correct knowledge of the absolute loss in irrigation of interest to several branches of the Engineering Profession, but also to public officials, legislators, and the public. Therefore, the Duty of Water Committee of the Irrigation Division in selecting the subject of this report has struck upon a very live topic, as well as one which embodies the essence of its work.

The term, "consumptive use of water," although not strictly involving a correct use of the word, "consumptive", has been more or less accepted by those interested in the subject. Like "duty of water" and many other technical terms it may be considered a distinctive identification mark which common usage has sanctioned and the profession has accepted.

The proposed definition of "consumptive use" the writer believes is sufficiently comprehensive, with the possible exception that it does not recognize either double cropping or intercropping. Both are practiced in California, particularly in Southern California and the Imperial Valley, and the annual consumptive loss per acre is materially increased thereby, especially in the case of double cropping. In order to recognize the practice in the definition, the words, "or crops", might be added after the word, "crop".§

* Discussion of the Report of the Duty of Water Committee of the Irrigation Division on the Consumptive Use of Water continued from September, 1928, *Proceedings*.

† Cons. Hydr. Engr., San Francisco, Calif.

‡ Received by the Secretary, September 24, 1927.

§ *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 216.

The mathematical form for analysis of consumptive use data is somewhat confusing at first reading, but its use is probably justified from the standpoint of completeness and simplicity. The writer has followed this method satisfactorily for many years in working out individual cases, both for determination of consumptive use and for equating ground-water absorption and discharge for valleys of the closed-basin type.

Net Consumptive Use.—The practicability of determining working values for net consumptive use, U, by means of tanks or potometers, is dependent on reproduction of natural conditions. The water supply is under control and is supplied more regularly and uniformly than is possible in practice and in more exact synchronism with plant requirements. This probably tends to increase rather than decrease evaporation and plant transpiration. Also, there is more complete exposure of the plant to light and wind, and there may be the absence of the vapor blanket present under field conditions. The tank in relation to the field is in fact the counterpart of the small land evaporating pan as compared with the lake or reservoir. The work of Sleight, at Denver, Colo.,* and of the writer in Owens Valley, California,† has shown conclusively that the evaporation from a small land pan considerably exceeds that from a lake or reservoir. Similarly, it is to be expected that the transpiration from a crop growing in a small tank not surrounded by growing vegetation of a similar character will exceed that from a cropped field. The conditions surrounding tank experiments should be ascertained before placing dependence on the results.

The method by field plot measurements on the other hand, while it has many advantages over the use of tanks has the disadvantage that draft on the ground-water and deep percolation loss are both unmeasurable. In the effort to avoid the former, experimenters have endeavored to select sites having great depth to ground-water, while in the case of the latter, small and frequent irrigations have been given in the hope that deep percolation loss would be reduced to zero. In each determination of U presented by the Committee there is more or less uncertainty whether or not deep percolation occurred and whether its value is really so small as to be negligible. Without more convincing proof, it is open to doubt whether in all cases deep percolation can be considered as negligible even with a deep water-table.

Farm Consumptive Use.—The conditions under which farm consumptive use (U_f)‡ occurs, differs but little from that of net consumptive use from field plots. The elements of difference are principally with respect to the application of water in accordance with actual farm practice rather than under experimental control, and frequently a shallower depth to the water-table. As is to be expected, the measured values of U_f, are generally greater than those for U determined at field plots. This is due to the inclusion of deep percolation in the former.

Table 10 assembles all comparable values presented in the Committee's report, those of Lewis in Snake River Valley being omitted because of lack

* "Evaporation from the Surface of Water and River Bed Materials," *Journal of Agricultural Review*, Vol. X (1917), pp. 229–230.

† *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), pp. 176–181.

‡ *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 217.

of data regarding soil moisture and crop season rainfall. A striking feature is the wide variation in apparent values of U and U_f. This is probably due both to errors of measurement and to difference in crop-growing conditions. Table 10 serves to emphasize the conclusion of Kisselbach that "there is no such thing as a definite water requirement which is constant for any kind of crop",* and points clearly to the fact that the acceptance and use of U and U_f data on any basis other than as an aid to judgment are not justified.

TABLE 10.—COMPARISON OF CONSUMPTIVE USE AS DETERMINED FOR FIELD PLOTS (U) AND FOR INDIVIDUAL FARMS (U), IN ACRE-FEET PER ACRE.

Crop.	Field Plots.				Individual Farms.		
	Widtsoe, Cache Valley, Utah.*	Snelson, at Brooks, Alberta.†		Widtsoe and Harris, at Utah Agricultural Experiment Station.‡	Hemphill, in Cache La Poudre Valley, Colorado.§		Israelsen and Winsor, in Sevier Valley, Utah.‖
		Minimum.	Maximum.		Average yield.	Maximum yield.	
Sugar-beets	2.5	....	....	2.4	3.0	3.0	3.1
Potatoes	2.2	....	....	2.2	3.3	4.2	2.6
Alfalfa	2.3	1.00	2.62	4.6	3.7	5.8	3.4
Corn	2.5	....	....	2.2	....	....	....
Wheat	2.4	0.85	1.82	2.5	2.1	3.1	....
Oats	2.5	0.72	1.75	3.1	1.4	3.6	....

* Crop season rainfall, 0.42 ft.; draft on soil moisture, 0.1 ft. for corn to 0.83 ft. for alfalfa.

† Crop season rainfall not stated; draft on soil moisture measured, but not stated.

‡ Depth of soil moisture and rainfall, 0.86, 0.52, 1.25, 0.52, 1.15, and 0.81 ft., respectively.

§ Crop season rainfall, 0.92 ft.; draft on soil moisture, 0.30 ft.

‖ Crop season rainfall, 0.30 ft.; draft on soil moisture, 0.30 ft.

The unusually large values for U_f found by Hemphill† in Cache la Poudre Valley, where the crop season rainfall was 0.92 ft., or more than twice that in any other location, raises a question as to the propriety of adding the full depth of the observed rainfall. The writer's own observations and a study of the observations of others lead to the conclusion that most or all of the summer precipitation in valley parts of semi-arid and arid regions is lost, either through surface run-off or immediate evaporation from foliage and the soil; and that very little percolates to a depth sufficient to make it available to any but the most shallow rooted crops. In view of this condition the addition of the total rainfall for the crop season does not carry out the purpose of the Committee, which was to arrive at basic values capable of use in any locality. It is believed that this item needs experimental study and that, pending the availability of further data, consumptive use values which include total rainfall should be used with caution. This suggestion obviously applies to consumptive use in all its phases—basic, farm, project, and valley.

Project and Valley Consumptive Use.—The determinations of "project use" (U_p)‡ and "valley consumptive use" (U_v),§ as defined by the Committee,

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 219.

† *Loc. cit.*, p. 225.

‡ *Loc. cit.*, p. 217.

§ *Loc. cit.*, p. 218.

are far from simple problems. Unlike U_f, in which the ultimate disposition of D_f is immaterial, in the case of U_p and U_v it is necessary to trace D_p and D_v at least to the extent of ascertaining the quantity returning to stream channels and the evaporation and transpiration loss from non-cropped land. Both U_p and U_v should be less than U_f, since a portion of D_f is accounted for where larger areas are considered. In the case of the project the unaccounted portion is that continuing as percolating water beyond the project boundary to appear later in a surface stream or pass into the atmosphere from an area of shallow ground-water. For valley consumptive use the unaccounted portion, if any, passes from the valley as sub-surface flow. If there is no sub-surface flow, as in the case of a closed geologic basin, U_v should be equal to U. The complications surrounding the determination of the values of U_p and U_v arise, first, from the inability to see the geologic formations beneath the surface of the ground; second, from the hidden course and slow movement of deep percolating water from irrigated land; and, third, from the mingling of such water with percolating water from other sources. An analysis of the Committee's report shows that the proposed method of determining U_p and U_v-values is still in a formative state and that the complications which are encountered in practical application are being dealt with either at arm's length or not at all.

The earlier investigators attacked the problem in its simplest form and attained roughly approximate results. Later investigators, recognizing the omissions and assumptions, have introduced refinements. The Committee's report is the latest step in this process and represents a notable advance. It is to be recognized, however, that practically all investigators have thus far approached the subject from the viewpoint of irrigated agriculture. From this background, the basic units of time and area have been the crop year and the cropped area. These units are essentially artificial and until natural units are adopted the concepts must continue to be in terms of special cases.

The next step forward is the adoption of the climatic year, instead of the crop year; and the topographic or geologic unit of area, instead of the project or project group area. The following examples will serve to illustrate the advantages of these units. In most localities where irrigation is practiced the crop year covers six to eight months, or from one-half to two-thirds of the full year. Due to the slow movement of ground water there is a considerable lag between the diversion of water from a stream and its return as seepage or spring flow. Water which passes beyond the plant roots joins the general ground-water body below (thereby raising the water-table), and remains in sub-surface storage until the water-table returns to its initial level. The lag period may vary from a few weeks to five or six months, depending on the character of the subsoil, the relative elevation of irrigated land and adjacent drainage channels, and the distance of the irrigated land from drainage channels flowing direct to the main stream. Observations which do not extend over the lag period fail to indicate the amount of delayed return reaching the outlet stream or evaporated from moist non-cropped soil during the non-irrigation period, and hence give consumptive use values which are too great. The writer's observations indicate that in certain cases the degree of error from this source may be very large and that it is seldom negligible.

Another illustration in which the scope of the Committee's analysis might be broadened is with respect to area. Consideration of inflow and outflow has been limited to the cropped area and the immediately adjacent land from which evaporation or transpiration from natural vegetation occurs. The attempt has been made to list the H and R-factors* with respect to a single project area or group of project areas within a valley. This procedure has given rise to difficulties of measurement. For instance, it is impossible in practice to measure the sub-surface flow into the crop area. This is recognized by the Committee in commenting on the many valleys of the Southwest where inflowing ground-water is important.† The statement is made that, in such instances, it is "impractical to obtain dependable measurements of project or valley water supply."

Although not mentioned, the same difficulty exists with respect to sub-surface flow from the cropped area. Some of this may appear as seepage or spring flow and be measured as outflow, but, on the other hand, there may be loss by evaporation from shallow water-table outside the project area, or sub-surface outlets other than the adjacent stream. There may also be storage accumulation as represented by rise of the water-table in non-cropped areas outside the project. By approaching the problem from the aspect of a topographic or geologic unit, however, these difficulties largely disappear.

The illustrations are sufficient to indicate the errors into which the investigator may fall if the methods outlined by the Committee are his only guide. It is not the writer's purpose to outline a method to replace that of the Committee, for the latter is well adapted for use in many cases, but rather to suggest a broader foundation for analyzing the individual problem more comprehensively and if necessary to modify the Committee's method to fit the case.

Since about 1910 there has been a notable accumulation of knowledge in the field of ground-water hydrology. The existence has been shown of many natural ground-water reservoirs occupying geologic basins more or less co-extensive with topographic valleys.‡ The source, movement, and ultimate disposition of the ground-water in these basins have been traced and methods have been developed for quantitative measurement of inflow and outflow.§ These methods have now become sufficiently standardized for practical use in determining the dependable yield from ground-water sources.

In 1926 the City of Los Angeles, Calif., completed a program of ground-water development in the Independence Region of Owens Valley the extent of which was originally planned from the results of a quantitative ground-water study made under the direction of the writer from 1908 to 1911. The development work, consisting of wells and pumping plants, cost approximately $250 000 and yields an auxiliary supply to the Los Angeles Aqueduct System of more than 100 sec-ft. The same investigational methods have also been applied to the solution of other problems. It is the writer's conviction that

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 215.

† *Loc. cit.*, p. 220.

‡ "Occurrence of Ground-Water in the United States," by O. E. Meinzer, *Water Supply Paper 489*, U. S. Geological Survey, pp. 291–303, 1923.

§ "Water Resources of Part of Owens Valley, California," by Charles H. Lee, *Water Supply Paper 294*, U. S. Geological Survey, 1912.

the methods of ground-water hydrology used in making quantitative determination of ground-water supply have direct application to the problems of project and valley consumptive use. They will serve to point out unsuspected items in the inflow-outflow equation, to evaluate unknown factors, and to make possible a solution in valleys with inflowing ground-water which is claimed to be impractical by the Committee.*

In adapting the methods of ground-water hydrology to the measurement of consumptive use, the first step is a geological survey to determine the character and extent of the impervious bed-rock basin, the limits of the hydrologic water-shed, the character and distribution of the porous alluvial fill, and the probability of sub-surface inflow or outflow. A more or less complete water-table map with typical fluctuation graphs covering the annual cycle should also be available, and a general knowledge of local precipitation, run-off, stream channels, irrigated land, seeped or swamp land, springs, etc. With such information it should be possible to list all ground-water sources and outlets and to ascertain whether there are any negligible or compensating items so that the problem can be reduced to its simplest terms.

Three general types of valleys are met in consumptive-use determinations:

1.—There are those without underlying alluvial fill in which the bed-rock formation is covered with a thin veneer of decomposed rock and soil in place. This type seldom occurs in practice. If the bed-rock is non-porous rock, such as granite, the valley can be treated under one of the other two classifications. For a porous, fissured, or cavernous bed-rock, the measurement of consumptive use may be impracticable.

2.—There are valleys of the "open-basin" type in which the underlying bed-rock basin is filled with porous alluvial material having one or more areas of direct contact with outside porous formations which may extend even beyond the topographic drainage area. This type also includes valleys where the bed-rock is leaky, due to the occurrence of porous or soluable bed-rock, such as open sandstones, limestones, etc.

3.—There are valleys of the "closed-basin" type, in which the porous alluvial fill has no connection with outside formations, being completely enclosed by non-porous bed-rock bottom and sides continuous with the surrounding hills or mountains.

Proposed Method of Determining U_v.—To illustrate the application of the proposed method of determining valley consumptive use, the various quantities will be defined and equations formulated. The period of time for which equations are set up is the climatic year and may be October 1 to September 30, November 1 to October 31, or January 1 to December 31, depending on local precipitation and run-off conditions.

The area, in the case of a closed geologic basin, is the superficial area of the porous alluvial fill, usually extending to the margin of the valley at the base of the surrounding hills or mountains. For an open geologic basin, the porous material of which extends beyond the margin of the topographic valley, the limits of the area may be chosen with respect to practicability of

* "Quantitative Methods of Estimating Ground-Water Supplies," by O. E. Meinzer, *Bulletin*, Geological Soc. Survey, 1912.

measuring or estimating the sub-surface inflow and outflow, although they should be restricted as much as possible.

The list of quantities has been designated by symbols which correspond wherever possible to those used by the Committee, as follows:

For annual inflow:

I_v = measurable surface stream flow, in acre-feet per cropped acre, entering the valley at the margin, including both main and lateral streams.

F_v = measurable or estimated sub-surface flow, in acre-feet per cropped acre, entering the valley at the margin.

P_v = the portion of direct precipitation falling upon the surface of the porous valley fill, which is not immediately evaporated and contributes to direct surface run-off, soil moisture, or ground-water, expressed in acre-feet per cropped acre.

For annual outflow:

R_v = measurable surface stream flow, in acre-feet cropped acre, at valley outlet.

D_v = measurable or estimated sub-surface flow, in acre-feet per cropped acre, escaping from valley, usually at the lower margin, although possible at other points.

U_v = evaporation and transpiration from cropped valley land, in acre-feet per cropped acre.

E_v = evaporation and transpiration loss from non-cropped valley land and natural vegetation, in acre-feet per cropped acre.

For annual sub-surface storage:

G_v = addition to or withdrawal from valley ground-water storage, in acre-feet per cropped acre.

S_v = addition to or withdrawal from valley soil water, in acre-feet per cropped acre.

The relation upon which equations are based is the annual equality for any valley of inflow and outflow with appropriate correction for sub-surface storage in the porous valley fill.

Expressing this equation by symbols:

$$I_v + F_v + P_v = R_v + D_v + U_v + E_v \, (G_v + S_v)$$

Solving for U_v, there results the general equation:

$$U_v = I_v + F_v + P_v - R_v - D_v - E_v \, (G_v + S_v)$$

For valleys of the closed-basin type, this reduces to the form:

$$U_v = I_v + P_v - R_v - E_v \, (G_v + S_v)$$

In arid regions where direct precipitation does not contribute appreciably to surface run-off, ground-water, or soil moisture, the equation reduces still further to the form:

$$U_v = (I_v - R_v - E_v) \, G_v$$

If the area of cropped land is of minor importance and it is desired to determine losses from natural vegetation, the last equation may take the form:

$$E_v = I_v - R_v - U_v \, G_v$$

To evaluate E_v in terms of acre-feet per acre of natural vegetation and seeped land, it should be multiplied by the ratio of the area of cropped land to that of natural vegetation.

The I_v and R_v-quantities are capable of measurement by ordinary stream-flow methods. The F_v and D_v-quantities are difficult and expensive to measure, but as they are ordinarily small in amount, they can often be estimated from data available from geological and water-table surveys. For closed basins they are zero.

The value of P_v although difficult to determine in semi-arid and humid regions is capable of measurement. Direct addition to run-off from precipitation on the valley floor can be determined from continuously recording water-level gauges on streams. Immediate addition to ground-water can similarly be determined from shallow well records and porosity data. Soil moisture additions are easily determined by ordinary soil moisture tests made before and after storms. The extent to which it is advisable to go in making detailed observations to determine the distribution of direct precipitation upon the valley floor is a question to be answered in each individual case.

The value of E_v is usually quite small relative to U_v and can be determined with sufficient accuracy from a knowledge of the area of moist land and natural vegetation, from local water evaporation, and from the results of observations in other localities.*

For determination of ground-water storage quantities, G_v and S_v, the standard methods can be used. Ground-water storage can be determined either from water-table fluctuations, as measured in shallow wells and test holes and porosity data, or by indirect methods, such as those used by Harding and referred to in the Committee's report.† It may also be eliminated from consideration where observations of other quantities covering several years are available and ground-water conditions as modified by irrigation have become stabilized. The variation in soil moisture from year to year is in most cases so small that it is negligible, and S_v may be eliminated altogether.

Many special cases are encountered in practice, which permit of modifications of the general equation, in addition to the simplifications stated. Harding, in his San Joaquin Valley work, follows essentially the method outlined by the writer; he adopts an interesting modification with respect to the ground-water storage correction. In his work in San Luis Rey and in other valleys in San Diego County, California, the writer used still other modifications, as suggested by the local synchonism in the periods of run-off and ground-water fluctuation.‡ Every locality and valley has its special conditions and in almost every case study will disclose important simplification in the application for the general formula.

Committee's Discussion and Conclusion.—The proposal to study consumptive use in conjunction with heat available during the crop year is logical.§ Temperature is by far the most important factor governing evaporation, the

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 334–338.

† *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 233–236.

‡ "Geology and Ground-Water of the Western Part of San Diego County, California," by A. J. Ellis and Charles H. Lee, 1919, pp. 142–155.

§ *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 237.

direct straight-line relation between the two having been clearly shown by Sleight* and other investigators. Transpiration being a special form of evaporation is controlled largely by temperature as is also the plant life process of which transpiration is a phase. The straight-line relation between valley consumptive use and heat available during the crop year, which is indicated by Fig. 2,† is therefore a reasonable assumption. It should be pointed out, however (in view of the present uncertainties regarding consumptive use values as determined without the aid of a standard method by various investigators, as well as the limited range of available heat values to draw from), that at present neither the straight-line relation nor the equation of any line can be considered as established. The first requisite is dependable data; the second, sufficient data.

The writer's experience leads him to modify the Committee's Conclusion 4‡ "that measurement of project and valley consumptive use under some physical conditions is impractical", to read, "that measurement of project and valley consumptive use is possible under most physical conditions met in practice." Under the method proposed by the writer, the determination of valley consumptive use in the only case specifically excluded by the Committee as impractical, is made entirely feasible.§

The last conclusion of the Committee (6‖) would appear to be correct provided the data are accurate. The writer, however, does not believe that the data now available are sufficiently dependable to permit conclusions to be drawn of such far-reaching importance.

Results of Measurements in Owens Valley, California.—The Owens Valley, California, is a desert valley typical of the Great Basin. It lies at the eastern base of the Sierra Nevada Mountains with an elevation of about 4 000 ft. The run-off from melting snows on these mountains has provided water for the irrigation of approximately 50 000 acres of land in the valley as well as the major part of the supply of the City of Los Angeles. This valley, because of its physical and meteorological conditions and the wealth of hydraulic data available, offers opportunities for determining values of U_v and E_v. The writer has had intimate familiarity with it since about 1905 and presents the following from personal knowledge.

The valley is a closed geological basin from which the only natural escape for water in appreciable quantities is by evaporation and transpiration. Precipitation on the valley floor varies from 3 to 5 in. annually, most of which occurs during the winter months. The average annual depth of evaporation from a free water surface is 67 in. Many lateral streams fed by snows falling upon the eastern slope of the High Sierra enter the valley along its western border. Such waters as are not locally consumed in irrigation find their way into the Owens River which flows down the trough of the valley and finally empties into Owens Lake. The latter is without surface or sub-surface outlet

* "Evaporation from the Surfaces of Water and River Bed Materials," *Journal of Agricultural Research*, Vol. X, 1917, pp. 229–230.

† *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 241.

‡ *Loc. cit.*, p. 240.

§ *Loc. cit.*, p. 220.

‖ *Loc. cit.*, pp. 240–241.

and is a great evaporating pan from which all residual waters of the valley are dissipated into the atmosphere. Its fluctuations are an accurate index of meteorological and cultural changes in the valley above.

Owens Valley is divided into three nearly equal parts by two ranges of granite hills lying across the axis of the valley and extending from the Sierra Nevada Mountains almost to the Inyo Range on the east. These ranges are known as the Poverty Hills and the Alabama Hills. The upper part of the valley thus subdivided has been termed the Bishop-Big Pine Region, the middle part, the Independence Region, and the lowest subdivision, the Owens Lake Region. The alluvial fill of the valley in the narrow necks opposite the east ends of these ranges of hills is composed of fine material consisting of fine sand, silt, and clay. The hydraulic grade of the main valley is so flat that subsurface flow from one region to another is negligible. Each of these smaller valleys, therefore, constitutes a geologically closed basin from which the only escape for water is the surface flow of Owens River, or by evaporation.

When white men first settled in the valley, the higher lands were dry and sandy, supporting a sparse growth of desert shrubs. Stream borders and the broad bottoms of the Owens River supported meadow grass and willows. Green meadows also occurred along the valley borders in the vicinity of springs. Depressions in the valley floor supported coarse grasses, and occasional moist alkali areas occurred where little vegetation grew. Irrigation commenced in the early Sixties and by 1875 the available supply from the lateral streams was being led out in small ditches on to the adjacent lands for the raising of grain and general field crops. In the late Seventies the construction of large ditches to divert the waters of Owens River commenced. The irrigated area was rapidly extended, alfalfa being introduced in the Eighties. By 1905, the available flow had been completely diverted and since then no further important extensions have occurred.

The effect of irrigation has been to raise the water-table in the vicinity of irrigated land and to greatly extend the meadow-grass and alkali areas of the main valley floor. The area of Owens Lake, on the other hand, has decreased and its level has been permanently lowered. As an incident to the latter, the Owens River has cut back on a new grade, developing a new channel in the river bottom, varying in depth from 20 ft. near the margin of the lake to zero at a point northeast of Bishop, approximately 70 miles up stream. The new conditions brought about by irrigation, including the water-table and return water, had stabilized by 1905. The condition thus described existed prior to the diversion of water from Owens River through the Los Angeles Aqueduct, which commenced in 1913. The latter did not become important in its effect on Owens Lake until 1918.

*U and E_v in the Independence Region.**—The Independence Region, during the period of intensive water supply study from 1908 to 1913 prior to the completion of the Los Angeles Aqueduct, had a total area (from which soil evaporation and transpiration occurred) of 37 950 acres. This acreage was made up of 3 010 acres of irrigated land planted to alfalfa and general crops, and 34 940 acres of natural meadow, salt-grass, and moist alkali land. The

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 148.

irrigated area consisted of ten or more detached groups of ranches on as many small streams, making it impracticable to determine a value for U_v. The conditions were ideal, however, for determining E_v, a quantity which is usually indeterminate.

The water-table beneath most of the irrigated land was too far below the surface to furnish an appreciable supply to crops, and the Owens River did not receive measurable sub-surface accretions. The water absorbed by the porous alluvial fill found its escape by evaporation and transpiration from meadow, salt-grass, and alkali land moistened by capillary water drawn up from the water-table. There was also escape in the form of spring flow reaching the main drainage channel of Owens River. The average annual percolation into the porous material of the valley which reached the water-table, as determined by the writer from observations made during a three-year period, was as follows:

Source.	Acre-Feet.
Direct precipitation on mountain slopes between canyons...	19 700
Direct precipitation on outwash slopes.....	9 780
Seepage loss from lateral streams.....	57 100
Return from irrigation.....	13 000
Absorption from flood water.....	6 500
Total	106 080
Deduct spring discharge.....	22 400
Net consumed by meadow, grass, and moist soil.....	83 680

$$E_v = \frac{83\,680 \text{ acre-ft.}}{34\,940 \text{ acres}} = 2.40 \text{ ft.}$$

A check determination based on a detailed survey of depth to water-table and two years of tank observations of evaporation and transpiration from typical sods with differing depth to the water-table, resulted as shown in Table 11.

TABLE 11.

Depth to water-table, in feet.	Area, in acres.	Average depth to ground-water, in feet.	Transpiration-Evaporation.	
			Feet.	Acre-feet.
3	7 610	2.5	3.48	26 400
3 to 4	11 300	3.5	2.80	31 600
4 to 8	16 030	5.5	1.32	21 100
Total.....	34 940	....	2.26	79 100

It is believed that the second value of 2.26 ft. for E_v is closer to the truth than the first, as the detailed observational control was more complete. In the first set-up the only item based on systematic and detailed observation

was seepage loss from lateral streams constituting 54% of the total. The other items were in part based on estimates. All items involved in the second set-up were the result of observation. Unusual precautions were taken to have natural conditions for the tank experiments.

U for Alfalfa in Independence Region.—Observations were also made near Independence, on August 9, 1910, to determine transpiration losses from alfalfa in a typical field stand. The method was that used by certain European investigators—determining the rate of loss of water from freshly cut plants, on the assumption that this would approximately represent the rate of loss immediately prior to cutting. Four samples were cut from measured areas at different times during the day. These were weighed at short intervals until no further appreciable loss occurred. Visible wilting did not occur during the first 15 min. after cutting, and this period was used in making calculations. The results were reduced to a 24-hour average, using a factor based on standard transpiration experiments. The local growing season extends from April 15 to September 30, and the production of dry hay amounted to about 5 tons per acre with three crops. The samples used for observation were almost ready for the second cutting. Assuming that the average transpiring surface during the entire growing season was 50% of that on the day of the experiment, the total loss for the season would have amounted to 935 lb. of water per lb. of dry hay, or 3.4 ft. in depth. The total depth of water applied in irrigation on this field during the crop season was 16. ft.

This value for U is consistent with the quantity of water available and the results of tank experiments elsewhere. The method, however, involves unnatural conditions. The samples were dried at the margin of the field out of the main vapor blanket, and the ends of the stems were cut, thus interrupting the life processes of the plant. It is not believed that the results by this method are any more dependable than those obtained by the use of standard tanks.

U_v and U_p in the Bishop-Big Pine Region.—The moist area of the Bishop-Big Pine region, although including considerable meadow and salt-grass, is primarily irrigated land, and the physical conditions are favorable for determining values for U_v. The region as here considered extends from Pleasant Valley on the north to the Poverty Hills on the south, including both sides of the valley. Stream measurements are sufficiently complete for the six years, 1918 to 1923, to determine total inflow from various streams at the margin of the valley as well as outflow in Owens River at Charlie's Butte. Acreage data were also quite complete. The water-table was shallow and D_v negligible. The soils vary from those of a coarse sandy granitic character to black loam. Water was taken from Owens River by means of thirteen ditches, the annual net head-gate diversion varying in depth from 9 to 4 ft. on the land, depending on the position of the ditch on the stream and the available supply. The average annual duty for the larger ditches with earlier rights exceeded 6 ft. The length of the irrigation season averaged 213 days, from March 15 to October 15. The land was irrigated from the Owens River, as follows:

Crop.	Acres.
Alfalfa	8 174
Orchard	850
Annual crops	4 684
Pasture (not cultivated)	20 809
Total	34 517

Approximately, 6 340 acres had a supplemental supply from Bishop and Big Pine Creeks, and, in addition, 10 427 acres, mostly in alfalfa and general crops, were irrigated entirely from creek ditches.

The total area of irrigated land, moist soil, and water surface during the period, 1918 to 1923, based on surveys by the City of Los Angeles and the U. S. Reclamation Service is:

Land Under Canals and Ditches:

Irrigated (cultivated and pasture)	42.984	acres
Corrals, roads, etc.	1 590	"
Tule and willows	1 960	"
Water surface (canals)	450	"
Salt-grass and rabbit-bush	5 405	"
Total	52 389	acres

Land not Under Canals:

Meadow and salt-grass	2 465	acres
Water surface (river, creeks, and lakes)	930	"
Total	3 395	"
Grand total	55 784	acres

The daily inflow into the valley from Owens River and Bishop-Big Pine and Baker Creeks, and several large springs, is summarized in Table 12, together with the outflow at Charlie's Butte. The inflow figures also include estimates for Horton, Birch No. 1, McGee, Tinemaha, and Birch No. 2 Creeks, aggregating 10% of the total inflow. The period of observation, although including several dry years, was characterized by wasteful use, and it is believed that the results for net loss are representative of average conditions for this valley. The period of observation preceded any appreciable withdrawal of land from irrigation by the City of Los Angeles.

The total evaporation and transpiration loss of 168 600 acre-ft., if spread over an area of 55 784 acres, indicates a value for U_v of 3.0 acre-ft. per acre.

In explanation of the apparently large value of U_v, the following classification of moist lands in the Bishop-Big Pine Region, made by the U. S. Reclamation Service in 1920, is illuminating:

Seepage water on surface	5 450	acres
Water, 0 to 2 ft. below surface	24 480	"
Water, 2 to 4 ft. " "	11 250	"
Water, 4 to 6 ft. " "	13 720	"
Total	54 900	acres

This statement shows an unusually large percentage of badly seeped land from which a high rate of evaporation and transpiration loss occurs. Much of this seeped land is classed in the area already noted as "irrigated pasture". The effect on U_v of the wasteful methods of irrigation and the paralleling of earthen ditch systems at successive levels, one above the other, is apparent. The value tends to confirm the results obtained by Harding in Reno Valley where methods of irrigation and meteorological conditions are similar.* In both cases, U_v could be reduced materially by the reconstruction of the ditch systems and improved methods of irrigation.

TABLE 12.—INFLOW AND OUTFLOW FOR BISHOP-BIG PINE REGION OF OWENS VALLEY, IN ACRE-FEET.

Year.	Total inflow.	Outflow.	Difference.
1918	400 200	260 800	139 400
1919	355 500	197 600	157 900
1920	348 400	170 600	177 800
1921	345 300	166 900	178 400
1922	481 900	294 200	187 700
1923	341 500	171 100	170 400
Average	.....	.....	168 600

The change in area and specific gravity of Owens Lake between 1878 and 1909, during which period the thirteen ditches diverting water from Owens River were built and put into operation, affords a method for determining the value of U_p for the area in the Bishop-Big Pine Region covered by these ditches. The shrinkage of the lake surface and increase in specific gravity of the water with the corresponding reduction of evaporation, should represent the combined loss from the newly irrigated land. An element which might materially modify the result, however, is the extent to which the channeling of Owens River on the new grade established by the lower lake level has reduced the evapo-transpiration from the river bottoms. Prior to irrigation in the valley the Owens River spread out more or less over its bottoms in high water for a width of ¼ to ½ mile, and at all times had a shallow, meandering and frequently subdivided channel. The bottoms were thickly overgrown with willows and grass from which large losses occurred. This condition was greatly changed for a distance of more than 70 miles by the cutting of the new channel. The water level in the river and the adjacent water-table are both lower than formerly, and flooding occurs only in very wet years. Also,

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 230.

the river channel is narrower and more concentrated. This results in great decrease in the amount of vegetation and change in its character, with reduction of losses. The degree to which these changed conditions confuse the results for irrigation is not known, but the data will be presented for what they are worth, since the method is unique and may be applicable in other localities.

The changes which took place at Owens Lake have been carefully determined by the writer and are as shown in Table 13.

TABLE 13.—CAPACITY CHANGES IN OWENS LAKE.

Period.	Stabilized lake level, in feet.	Area of lake surface, in acres.	Specific gravity of lake water.	Annual depth of evaporation, in inches.	Total evaporation, in acre-feet.
1872-78.....	3 597	71 142	1.04	64.6	384 000
1909-15.....	3 576.5	61 600	1.093	61.1	313 500
Difference....	20.5	9 542	.....	....	70 500

The lake being without outlet, is highly mineralized, and its specific gravity has exceeded that of sea water even at the highest historic lake level. A long record of lake levels is available, together with accurate area and capacity curves of the lake basin, a "specific gravity-lake level" curve and a "specific gravity-evaporation" curve.*

The value of 70 500 acre-ft. for the reduction in annual evaporation from Owens Lake between 1878 and 1909 the writer regards as reasonably accurate. If all the reduction was occasioned by the increased area of irrigated and seeped land, the value of U_p for the net area irrigated from river ditches of approximately 34 517 acres, is 2.00 ft. As this value is considerably less than U_v as determined previously, it is possible that the shrinkage in flooded river bottom-lands along 70 miles of river channel, due to channel cutting, should also be taken into account in order to get the true value of U_p. The detailed conditions along the river as they existed prior to 1878 are so difficult to determine, however, that it is doubtful if it will ever be possible to get an accurate value for the correction.

Results of Measurements in San Luis Rey Valley.—The San Luis Rey Valley is a coastal valley on the Southern California coast about 50 miles north of San Diego. It extends from the seashore into the mountainous interior for a distance of more than 20 miles, varying in width between steeply sloping hills from 300 ft. to nearly 2 miles. The grade of the valley floor averages 20 ft. per mile, being least near the coast. The run-off from precipitation, mostly in the form of winter rainfall on a drainage area of 565 sq. miles, discharges into the valley from various tributaries and flows thence to the Pacific Ocean. The flow of the San Luis Rey River is confined to storm periods during the winter months, except at the head of the valley where there is a small living stream.

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 333.

The bed-rock formations underlying and surrounding the valley are relatively impervious and no leakage into them occurs from the porous alluvial fill immediately underlying the valley floor. The fill of the main valley has a superficial area of 7 646 acres and its maximum depth varies from 100 to 200 ft., thinning out to zero at the margins of the valley. The material is largely water-bearing sand with gravel at lower levels. The water-table stands near the surface, the depth varying at different points from zero to 10 or 12 ft. The average annual fluctuation of the water-table is 3 ft., the source of supply being percolation from the flow of San Luis Rey River and direct rainfall. The escape is by evaporation from moist sand and soil, and by transpiration from meadow and salt-grass and from willow, alder, cottonwood, and sycamore trees. Underflow, as measured, was found to be so slow that sub-surface escape to the ocean is negligible. The total area from which evaporation and transpiration takes place is 6 640 acres.

The annual precipitation on the valley floor varies from 10 in. at the coast to 12 to 16 in. at the head of the valley above Pala. Most of this falls during the winter months. The annual depth of evaporation from large bodies of water varies from 58 in. near the coast to 66 in. farther inland.

The writer made an intensive study of the underground water supply of the San Luis Rey Valley during the years, 1913 to 1916, and from these data it is possible to determine E_v for the valley. During the period of observation the area of cultivated crops did not exceed 4% of the total evapo-transpiration area, so that separate classification for irrigated land is unnecessary.

TABLE 14.

Item.	Location.	Character of vegetation.	Method of measurement.	Depth, in feet.
U.....	Independence Region of Owens Valley.	Alfalfa.	Weighing freshly cut plants.	3.40
K_v.....	Independence Region of Owens Valley.	Natural meadow, salt-grass, and moist alkali soil.	Large-scale measurements of percolation.	2.40*
E_v.....	Independence Region of Owens Valley.	Natural meadow, salt-grass, and moist alkali soil.	Soil tanks and water-table survey.	2.26*
U_v.....	Bishop-Big Pine Region of Owens Valley.	Irrigated crops, pasture and seeped land.	Large-scale measurements of H_v, R_v, and E_v.	3.00†
U_p.....	Bishop-Big Pine Region of Owens Valley.	Irrigated crops and pasture.	Shrinkage in area and increase of specific gravity of Owens Lake	2.00‡
E_v.....	San Luis Rey Valley.	Natural meadow, and salt-grass land, willows, etc.	Large-scale measurement of H_v and R_v.	2.19*

* Based on total acreage of natural vegetation plus very small cropped acreage.

† Based on total cropped acreage plus irrigated pasture, seeped land, and water surface.

‡ Based on total cropped acreage plus irrigated pasture. No correction for reduced river-bottom area.

There being no leakage and negligible underflow to the ocean, the valley may be considered a closed basin. The annual fluctuation of the water-table is within definite limits and storage fluctuation due to wet and dry years is small. With these conditions, E_v may be considered as equal to the average annual volume of water reaching the water-table by percolation from run-off and direct precipitation. Without going into details which are fully set forth elsewhere,* it was found that the average annual recharge based on three

* *Water Supply Paper 446*, U. S. Geological Survey, pp. 149–153 (1919).

years' records, was 14 450 acre-ft. This recharge, spread over 6 640 acres of moist land, gives a value of E_v of 2.19 ft. A check determination made independently and based on a detailed classification of moist areas and the application of the principles controlling soil evaporation and transpiration, as developed in the Owens Valley work and elsewhere, but using local rates for water evaporation, indicated an annual loss of 13 980 acre-ft., or a value for E_v of 2.10 ft.

Table 14 summarizes the values for U, U_p, U_v, and E_v, as found for the Owens and San Luis Rey Valleys.

M. R. LEWIS,* M. AM. SOC. C. E. (by letter).†—The consumptive use of water is a very definite element of great importance in irrigation, which has long been recognized in regions such as Southern California, where all the available water has been pumped from the pervious fill of closed valleys. More recently its importance has been recognized in studies of the water supply available for whole valleys rather than for individual projects.

The very great increase now taking place (1929) in pumping from wells for drainage and to supply supplementary water is certain to make the determination of consumptive use of vital importance in many areas. Heretofore, the water supply for an irrigation project has generally been based on the quantity of water required to be delivered to the farmers' head-gates plus transmission losses. If, in the course of time, return flow to the stream by natural seepage or by gravity drains has warranted, and if lower lands were available, the irrigated area has been extended. In places where pumps are used for drainage the water becomes available for use on the lands which are being drained or on other lands at approximately the same elevation. In every such case the question arises as to whether there is an excess of either land or water. Old data on the duty of water do not apply. For these reasons, the report‡ of the Committee is especially opportune.

The writer wishes to suggest a few additions to and modifications of the detailed statements of the report.

The terms, D_f, D_p, and D_v, all include seepage losses from ditches and canals in the respective areas.§ A specific statement to this effect would add to the clarity of the report.

If in determining H_p any negative value of (c), the depletion of groundwater, is considered, it should not also be considered in the determination of R_p in (b), such deep-soil percolation losses as contribute directly to a measurable rise in the height of the water-table.‖

The writer believes that it would be better to define U_p as being equal to $\frac{W_p}{A_p}$ and V_p as equal to $\frac{W_v}{A_v}$.‖ Then Equations (4)¶ and (6)¶ become,

$$U_p = \frac{A_{cp}}{A_p} U + D_p + E_p \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

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† Received by the Secretary, January 2, 1929.

‡ *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 214.

§ *Loc. cit.*, pp. 217–218.

‖ *Loc. cit.*, p. 217

¶ *Loc. cit.*, p. 218.

and,

$$U_v = \frac{A_{cv}}{A_v} U + D_v + E_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which, A_{cp} and A_{cv} are the cropped areas in the project and valley, respectively; A_p and A_v are the total areas in the project and valley, respectively; and W_p and W_v are the volumes of water, in acre-feet, consumed in the project and valley, respectively.

If these equations are used, the error in defining the word "consumptive"* would not occur and Equations (5)† and (7)‡ would be numerically correct.

The writer is glad to see so many different studies of consumptive use brought together in a single paper. The conditions leading to inaccuracy inherent in plot determinations of evapo-transpiration are realized. It is believed, however, that the difficulty of preventing or even of recognizing deep percolation losses in plot experiments is so great that it overshadows the errors in large-scale pot experiments on the one side and in valley consumptive-use studies on the other.

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* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 216, footnote.
† *Loc. cit.*, p. 218.
‡ *Loc. cit.*, p. 219.

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PAPERS AND DISCUSSIONS

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THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE

Discussion*

By Charles Terzaghi, M. Am. Soc. C. E.†

Charles Terzaghi,‡ M. Am. Soc. C. E. (by letter).§—This paper has called forth discussions from nineteen engineers in the United States, Russia, India, and China, representing a wide range of experience. While demonstrating the widespread interest in foundation problems, the discussion also disclosed the fact that few readers had an opportunity to consult the publication from which the writer's Statement (*a*)‖ was reproduced. For this reason, and on account of the condensed form of the statement, there was a tendency to misinterpret its meaning.

It has been previously emphasized that "cohesion" and "internal friction" are by no means simple properties. They depend not only on the character of the individual grains (shape, size, nature of their surface) but, to a variable degree, on the density, the water content, the pressure exerted by the surface tension of the water, and the hydrodynamic stresses which are associated with changes in load conditions. For very fine-grained soils, if the water content decreases due to evaporation, the cohesion increases rapidly. If friction tests are made with such soils so that during the test the water content of the specimen remains unaltered, results are obtained that are very different from those furnished by tests made on identical specimens enclosed between permeable plates.

Under these conditions it is self-evident that the terms, "cohesion" and "internal friction", represent abstractions and the validity of any statement which involves such abstractions necessarily depends on the validity of the simplifying assumptions on which the statement is based. This is obviously

* Discussion of the paper by Charles Terzaghi, M. Am. Soc. C. E., continued from January, 1929, *Proceedings*.

† Author's closure.

‡ Prof., Foundation Eng., Mass. Inst. Tech., Cambridge, Mass.

§ Received by the Secretary, January 7, 1929.

‖ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2268.

also true of the writer's Statements (*a*), (*b*), and (*c*).‡ These refer to two different materials: (*a*) Those for which the bearing power depends essentially on cohesion, internal friction representing a negligible item; and (*b*) perfectly cohesionless materials. The validity of the statements is limited by the following assumptions:

(1) The shapes of the loaded areas are geometrically similar.
(2) The term, "settlement", refers strictly to the total downward movement measured after the structure has reached a state of complete rest.
(3) The comparison between the settlement of loaded areas of different size should be confined to those produced by unit loads which are less than about one-half the ultimate bearing capacity for the smallest area.
(4) Hooke's law need not necessarily be accepted, but Poisson's ratio is assumed to be a constant.

Assumptions (1) to (4) apply to both Cases (*a*) and (*b*). The following additional assumptions apply to Case (*a*):

(5) The mechanical properties of the loaded material (relation between stress and strain and the compressive strength of the material) are practically independent of time and of the depth below the surface.
(6) At any point of the underground, failure occurs as soon as the difference between the greatest and the least principal stress exceeds the compressive strength of the material.

For Case (*b*), Assumptions (5) and (6) should be replaced by the following:

(7) The mechanical properties of the material (relation between stress and strain) are practically independent of time and of the depth below the surface.
(8) The loaded material is incompressible; that is, the settlement of the load is merely due to the deformation, but not to the volume change of the loaded material. This condition combined with Assumption (4) leads to the following consequence: If all the principal stresses acting on a prismatic element of the material are increased by multiplying their intensity by equal amounts, the element changes neither in shape nor in volume, irrespective of the intensity of the stresses.
(9) At any point of the underground, failure occurs as soon as the ratio between the greatest and the least principal stress exceeds a definite limiting value. This condition represents the most important distinguishing feature between Cases (*a*) and (*b*); it also represents the fundamental assumption on which all earth pressure theories of perfectly cohesionless materials since Coulomb were based and may be considered almost strictly correct.

The subject content of Statement (*a*) is graphically illustrated in Fig. 50. Curve C represents the result of a loading test performed on an area (circular, square, or rectangular) with a width of 1 ft., supported by either one of the two ideal materials described by the writer. Curve C_1 shows the characteristic results of a loading test performed on an area with a geometrically similar shape, but with a width of 3 ft. supported by the first one of the ideal materials, and Curve C_2, for the second one. The term, "ultimate bearing capacity", refers to the pressures p_1 and p'_1 respectively, at which a very small increase

in load produces a very important increase in settlement. The settlements produced by equal unit loads acting on areas of various sizes will be termed the "equivalent settlements" of those areas. Using these terms, the laws illustrated by Fig. 50 would be:

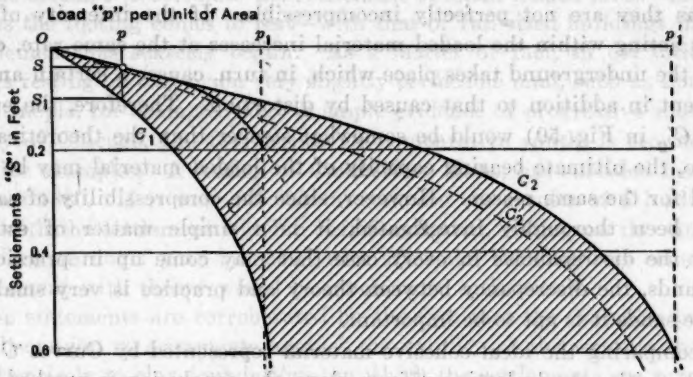

FIG. 50.—SETTLEMENT CURVES FOR LOADED AREAS.

(*A*) If the ultimate bearing capacity of the loaded ideal material is essentially due to cohesion, the results of the loading tests performed on areas with a width, *d*, equal to 1 ft. and 3 ft., respectively, would be represented by Curves *C* and C_1 (Fig. 50). Under a load of *p* per unit of area, which is assumed to be much smaller than p_1, the settlement, S_1, of the large area is approximately three times that of the smaller area, *S*; or, in other words, the equivalent settlement increases with increasing values of *d*. On the other hand, at a unit load of p_1, a small increase of the load produces for both areas a very large increase of the settlement, because both curves, *C* and C_1, bend steeply downward as soon as the load approaches p_1. Therefore, it is correct to state that the ultimate bearing capacity is independent of the size of the area.

(*B*) If the loaded ideal material is perfectly cohesionless, a similar pair of loading tests would be as plotted in Curves *C* and C_2 (Fig. 50). For a load of *p* per unit of area, the settlement, *s*, is practically independent of the size of the area, provided *p* is substantially smaller than p_1. On the other hand, the small area practically yields under a load of p_1 per unit of area, while the larger area must be loaded with $p'_1 = 3\,p_1$, in order to produce ultimate failure. Hence, it is claimed that the ultimate bearing capacity of the ideal, perfectly cohesionless materials (Law *B*) increases with the width of the area.

There was some doubt in the minds of most of the discussors as to whether Laws (*A*) and (*B*) were derived from theory or from experiments. To remove this doubt, it should be stated that the conclusions were obtained entirely by theory which was based on Assumptions (1) to (9), inclusive. As a matter of fact, if the validity of these assumptions is granted, Laws (*A*) and (*B*) need no proof, because they represent a simple consequence of the laws of similitude. Hence, any inconsistency between theory and practice must be ascribed to the properties of the material being at variance with those of the assumed, ideal materials.

If the properties of any soil conform reasonably well with the assumptions of theory, the laws represented by Fig. 50 must be valid. However, a comparison between the ideal materials and the actual soils leads to the conclusion that clean, conhesionless sands differ from the ideal cohesionless material inasmuch as they are not perfectly incompressible. If the intensity of all the stresses acting within the loaded material increases at the same rate, compression of the underground takes place which, in turn, causes a certain amount of settlement in addition to that caused by distortion. Therefore, the empirical curve (C'_2 in Fig. 50) would be somewhat steeper than the theoretical curve, C_2; also, the ultimate bearing capacity of the loaded material may be slightly reduced for the same reason. However, since the compressibility of sands has already been thoroughly investigated, it is a simple matter of estimating roughly the discrepancies in every case that may come up in practice. For dense sands, the discrepancy between theory and practice is very small, while for loose sands it is apt to be important.

By comparing the ideal cohesive material represented by Curves C and C_1 (Fig. 50), with actual plastic uniform clays, the conclusion is reached that all assumptions are fairly accurate, except for a certain increase in the cohesion of the clay caused by progressive consolidation of the loaded material. Yet this increase could not possibly account for the striking discrepancy between the settlements which should be expected for clay foundations from the results of loading tests and the actual behavior of buildings standing on such materials. Among the fundamental assumptions of the theory, there is just one which could possibly be made responsible for the apparent defect of the theoretical results. According to Assumption (2), the term, "settlement", refers strictly to the total downward movement until the structure comes to rest. In practice, the settlement observations on buildings are made, at best, a few years after the buildings have been constructed, and when the settlement is still taking place.

The time which has to pass until the settlement of a fair-sized building resting on clay would approximately conform to Laws (A) and (B) obviously depends on the time required for the excess water to drain out of the clay beneath the building. This time, in turn, depends on the permeability of the clay and can be estimated by the theories previously published by the writer.* Suppose, for instance, that the settlement due to pure compression of the clay beneath an area 1 ft. wide was, after 5 days, 0.5 in. According to theory, for an area with a geometrically similar shape, 20 ft. wide, subjected to exactly the same load per unit of area, it would take $5 \times 20 = 100$ days to settle through the same distance of 0.5 in., provided the excess water could not escape except through the exposed surface of the clay, surrounding the loaded area; or, in other words, provided that the bases of the loaded footings were water-tight. On the other hand, if the bases of the loaded footings were permeable, the settlement of both the small and of the large areas, after 10 days, would be approximately identical. However, for the small area the settlement would stop after it had reached during a period of several months a total of about 1.5

* "Erdbaumechanik", Wien, 1925.

in., while for the larger one it would continue with decreasing speed until after several generations it would reach an ultimate value of $1.5 \times 20 = 30$ in.

Thus, it seems highly probable that the apparent contradiction between theory and practice for clay foundations is due not to a defect of the theory, but to the attempt to correlate the settlement of small loaded areas—measured almost as the footing comes to rest—with that of full-sized buildings in which the settlement has scarcely begun. As a matter of fact, in old cities with buildings resting on clay or on very slightly permeable mud, such as London or Constantinople, for example, there is ample evidence of progressive subsidence. The City of Chicago, Ill., too, despite its comparative youth, is crowded with instructive examples. A section of Cambridge, Mass., has subsided more than 2 ft. since 1880. If it were possible to compare the results of a loading test on clay with the settlement which a raft foundation, resting on the same clay, had undergone not 1 year but 200 years after the building was constructed, the agreement with theory would be much more satisfactory.

These statements are corroborated by the fact that the examples presented in the discussion as arguments against the validity of the writer's statement, referred entirely to clay foundations, on which the settlements are retarded by the enormous resistance of the material against the escape of the excess water. No contradiction was found to exist between theory and practice for foundations of friable limestone, muck soils, silt, and sand, and other fairly permeable materials (see, for instance, the discussions of Mr. White* on limestone, Mr. Shaw† on muck soils, and Mr. Wing‡ on sand and gravel).

The bearing capacity of most of the more permeable soils commonly found in practice, for example, sands with traces of silt, or deposits of "muddy sands", is appreciably influenced by both cohesion and friction, the relative importance of cohesion depending on the composition and on the density of the material. For such soils, a loading test performed on an area 3 ft. wide should furnish a curve somewhere within the shaded area in Fig. 50, its shape and position depending on the degree to which cohesion participates in producing the bearing capacity.

Attention should also be called to the fact that Laws (*A*) and (*B*), or any rules for the interpretation of load test results, are based implicitly on the assumption that the soil is practically uniform and homogeneous to a depth at least equal to 1.5 times the width of the raft foundation, the settlements of which are to be predicted, or of the width of the area over which the footings are distributed.§ Since in practice this condition seldom exists, the field for successful application of the load-test method for determining the admissible bearing value of the soil is far more limited than most engineers seem to realize.

In his paper the writer quoted the rules illustrated by Fig. 50 (Laws (*A*) and (*B*)), merely for the purpose of demonstrating that no real contradiction exists between the different settlement observations quoted in the paper (Chi-

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1832.

† *Loc. cit.*, October, 1928, Papers and Discussions, p. 2313.

‡ *Loc. cit.*, January, 1929, Papers and Discussions, p. 84.

§ According to Assumptions (5) and (7), p. 950, strict validity of the laws requires uniformity of soil conditions to a depth equal to infinity.

cago, San Francisco, or the test results of Goldbeck and of von Emperger), and that the apparent contradiction between these test results disappear if they are "examined in the light of applied mechanics".* In all these cases, the properties of the loaded materials (hardpan in Chicago, succession of layers of clean or sticky sand in San Francisco), or the conditions under which the tests (by Goldbeck and von Emperger) were made, warranted the application of these principles (see Fig. 50). In contrast to this, no such application would be justified for correlating the results of a small-scale loading test on clay with the observations covering a period of a few months or years of the history of the settlements of a raft foundation resting on the same clay. In brief, the application of any rule to practical cases should be preceded by a careful investigation in order to determine to what extent, if any, the practical conditions agree with the fundamental assumptions of the rule. Due to the wide variation in soil properties no rules can be established that are valid for all soils.

The discussions deal essentially with two main topics—the *Engineering News* formula and the interpretation of the results of loading tests.

Among those who discussed the *Engineering News* formula, only one had anything to say in its favor. The others rejected it more or less emphatically, and without exception, on the basis of past experience. However, little attention was paid to what the writer considers by far the most important feature of the situation, namely, that the most general pile-driving formula (Equation (3)†), is fundamentally rigorous, provided one grants the validity of the basic assumptions. The most important of these assumptions concerns the identity of the static and the dynamic pile-driving resistance. The failure of the formula to give satisfactory results for pile-driving through clay or saturated silts is due merely to the fact that, in these cases, the conditions existing in practice fail to conform with the theoretical assumptions.

These remarks lead to the attitude of the discussors and of engineers in general toward the writer's statements as illustrated in Fig. 50. Some experienced engineers have accepted these statements as not being in contradiction with their own observations. Others, also, backed by broad personal experience, have rejected them. This fact should arouse the suspicion that conditions are not simple enough so that solutions can be found in the shape of rough and ready half empirical formulas. In a third group of discussions, attempts have been made to disprove the statements by proposing formulas which are claimed to agree better with practice.

These reminded the writer of the many discussions he has read about pile-driving formulas in past years. The method of approaching the problem was always the same. As often as a new formula appeared it was found invariably that the results furnished by it did not agree with many of the observed facts; therefore, it was argued that the formula must be modified. Nobody seemed to suspect that its deficiency might be due to the nature of an assumption in common to both the criticized and the proposed formulas. Hence, the discussion of the pile-driving formula revolved for half a century in a vicious circle.

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2268.
† *Loc. cit.*, p. 2274.

None of the discussors seemed to realize that the writer's statements stand as firmly "on their feet" as applied mechanics, provided the fundamental assumptions conform with the conditions existing in practice. The assumptions cover all the essential mechanical properties of the soils, except those which depend on time as, for instance, the compression of a clay or of a very fine saturated silt. In a similar manner, the assumptions on which Equation (3) is based consider all the essential elements of pile-driving, except the fact that, in many cases, the dynamic and static pile-driving resistance are not identical. Hence, it is justifiable to conclude that Laws (*A*) and (*B*) must be correct, except for cases in which the time factor partly or wholly upsets their validity for the brief period covered by the observations.

Instead of presenting this simple, but very important argument, the members of the third group of discussors merely tried to replace the laws, which have a limited but known range of validity, by others which are neither justified by rigorous analysis nor supported by a known range of validity. In doing so, this group repeated the mistake made by discussors who tried to replace the classical pile-driving Formula (3) by other more objectionable ones, instead of inquiring as to the fundamental cause of the observed disagreements.

These remarks also apply to discussors of the second group who attempted to question the soundness of the writer's statements solely on the ground of past experience. After realizing the limitations of theory, these engineers may ask: What then is the use for establishing rules at all? The answer to this question touches the most vital point of the science of foundations, and it cannot possibly be over-emphasized.

Settlements, as encountered in practice, depend as a rule on a number of circumstances, such as depth of foundation; presence of other structures in the vicinity of the new building; the occurrence of alternate layers of slightly permeable and markedly permeable strata; total thickness of the compressible deposits; relative position of the weakest spots of the underground with reference to the base of the foundation; and many others. In brief, the settlement of a structure is a function of so many variables that it would be hopeless to arrive even at crude and empirical conceptions of the causes of settlements unless systematic efforts are made to evaluate the relative importance of each one of the individual variables. This can be done merely by establishing a theory, not of the complex materials encountered in practice, but of a set of ideal materials with certain very simple properties, each one of which will serve as a means of studying one individual aspect of the real soils.

By such systematic analysis only can engineers gradually gain a deeper insight into the workings of actual soils. Since by necessity the very best analysis of that type falls far short of taking into account all the conditions that exist in practice, there always will be a certain gap between theory and practice, which can be bridged merely by experience. Considering the existence of this gap, the writer agrees heartily with Mr. Gow's warning* against readily accepting common formulas for use in actual practice. The writer even doubts whether formulas with a fairly general degree of validity could be

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 376.

established at all. Theory in connection with foundation engineering should be regarded merely as an essential supplement to experience, to be used as a tool for acquiring a deeper insight into the causes of observed foundation defects and for evaluating the relative importance of the factors in existence. Without a more profound analysis of phenomena encountered in practice, experience is doomed to remain the same accumulation of incoherent, meaningless facts which fill the space between the covers of many textbooks on foundation engineering. The time may come when a man with experience, but without a thorough theoretical training, will be considered almost as unfit to be trusted with a design of an important foundation as the theoretician who has had no experience. In structural engineering, this time came nearly a century ago.

The writer hesitates to accept Mr. Gow's advice concerning the *Engineering News* formula. If, as Mr. Gow states, it is a fallacy to rely too strongly on the results of individual pile tests, the application of the *Engineering News* formula seems to represent an even greater fallacy.

It is significant that Mr. Goodrich* abandoned the formula long before the writer had even heard about its existence. The observations made by Mr. Goodrich concerning the stress distribution over a square bearing plate should be an incentive for similar investigations of raft foundations resting on clay soils by means of Goldbeck pressure cells or similar devices.

The experience reported by Mr. Endersby† reminds the writer of a similar incident which occurred in Galveston, Tex. On the south side of the Island of Galveston, to a depth of about 50 ft., the underground consists of very fine sand mixed with powdered shells. Because of the very dense structure of this sand, it is considered impossible to drive piles to a depth of more than 15 or 20 ft. without smashing the pile-head. According to Colonel Schley, Government Engineer of Galveston, some piles driven with a steam hammer to refusal settled afterward several inches under a static load of less than 15 tons. This and the incident mentioned by Mr. Endersby are obviously decompression phenomena which can also be reproduced in the laboratory. If a layer of sand that is laterally confined is subjected to a rapidly applied pressure up to a certain value and then the pressure is kept constant (possibly by a testing machine), the counter-pressure of the sand against the piston first drops rapidly and then slowly to less than one-half the original reaction. In connection with this statement, it should be emphasized that there seem to be no records of conspicuous decompression formulas except for cases in which the pile-driving was done with a steam hammer that could apply pressure very rapidly. The intensity of the decompression phenomena may be considerably increased by the presence of a layer or pocket of very loose sand a short distance beneath the point of the pile. Since the number of known conspicuous decompression phenomena on piles is exceedingly small compared to the number of cases in which the piles behaved normally, such a possibility could at least be considered.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 377.
† *Loc. cit.*, p. 378.

Mr. Endersby's statement* that the result of the tests with three different hammers was "a good test for consistency of the [*Engineering News*] formula as applied to various types of hammers", should be accepted with caution. The satisfactory agreement between the results appears to be mere chance; otherwise, it would always exist. To mention one example among several on record: When driving the piles for the Mariotis Lake Bridge, in Egypt, three different pile-drivers were used: A steam hammer (weight, $R = 4\,400$ lb.; drop, $h = 3.28$ ft.); a hand-operated drop hammer ($R = 1\,320$ lb.; $h = 6.56$ ft.); and a steam drop hammer ($R = 4\,400$ lb.; $h = 3.28$ ft.). The points of all the piles penetrated the same stratum of sand.† The *Engineering News* formula furnished for the safe bearing capacity of these piles the average values of 66 800, 16 800, and 27 600 lb., respectively. If the *Engineering News* formula would furnish consistent results for piles driven into the same stratum with different types of hammers, these three values have been fairly identical. Mr. Endersby's statements show how a formula can acquire a good reputation by an act of Providence rather than by inherent merits. Since engineers very seldom have a chance to watch different types of pile-drivers operating simultaneously on a very homogeneous material, they are apt to base their opinions on an inadequately small amount of actual evidence.

Mr. Paaswell‡ justly emphasizes the fact that the New York Subway "is unquestionably the world's best laboratory for the study and illustration of all types of foundation problems". The same was true for the Panama Canal in the field of stability of earth slopes. Unfortunately, very few results of lasting value were derived from the Panama experiment because of lack of funds available for purely scientific research. In New York, the unique opportunity for systematic research seems by no means to be utilized to full advantage. The New York Subway remains a beautiful laboratory without equipment and without personnel. The profession should appreciate all the more the idealism of those few engineers engaged in subway construction who, by their own initiative, seriously endeavor to broaden technical knowledge on this subject by systematic experimentation.

Mr. Williams§ presents a theory based on three assumptions: (*a*) That settlement is merely due to compression of the underground; (*b*) that the total depth of soil compressed is limited by the depth at which the pressure intensity (per unit of area) drops below a certain limiting value; and (*c*), that the coefficient of compressibility, C, is independent of depth. Attention should be called to the fact that Assumption (*c*) could merely be considered approximately valid for materials with strong cohesion; for cohesionless materials, on conclusive, experimental evidence, it does not apply. Mr. Williams then computes the settlement of square areas with different widths, b. From Assumptions (*a*) to (*c*), it is evident that h (Fig. 18‖), increases in simple proportion with b. This fact is also expressed by Mr. Williams' formula,

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 378.
† According to Mr. W. Stross, in *Beton und Eisen*, 1908, p. 113.
‡ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 656.
§ *Loc. cit.*, p. 657.
‖ *Loc. cit.*, p. 659.

$h = \frac{b}{r}\left(\sqrt{\frac{w}{p}} - 1\right)$.* By introducing the value of h into the terms preceding this formula, the same results are obtained as those expressed by Curve C_1, Fig 50. However, instead of retaining h, Mr. Williams ceases to consider it as a function of the width, b, without attempting to justify his step, thus transforming it into a constant. This operation leads to the formula† $W = A\,p + L\,f$, which contradicts his theory. Since his final formula does not represent the result of the theory, it could be accepted merely as an empirical equation and the writer considers that the empirical evidence is too fragmentary to warrant such a step.

The example quoted by Mr. Williams‡ (the office building of the Michigan Central Railway Company in Detroit, Mich.) does not invalidate Curve C_1, Fig. 50. To conform with the assumptions of theory, the underground should be uniform to a depth equal to at least one and one-half times the width of the building. In contrast to this, from his own experience, the writer can state that the clay deposit of Detroit is far from being homogeneous. It consists of an irregular succession of layers of soft and stiff clay.

Furthermore, the observations quoted by Mr. Williams refer to a recently constructed building and settlements are not comparable as long as one of them still continues. For a building resting on clay foundations, the settlement twenty years after it was constructed is certainly not identical with that just after the building was completed. It may be many times more important. Thus, for instance, in Massachusetts, a short time after construction, the settlement of an important structure founded on plastic clay ranged between 0.1 and 1.1 in. One year later, the extreme settlements had increased to 0.6 and 2.8 in., respectively, and ten years later, they amounted to 1.0 in. and 7.6 in., respectively. In Chicago, conditions are not very much better.

Mr. Williams suggests reducing the discrepancy between the results of the loading test and the actual settlement by loading the soil around the test footing. To follow this procedure would mean an attempt to compensate for the effect of the time factor, which is absent, by adding a load factor. The importance of the time factor on the relation between settlements of small and large footings is exceedingly different even for the same clay, depending on the water content. Hence, it seems rather difficult to establish a reliable rule for computing the intensity of the compensating load which should be applied around the footing. Instead of trying to reduce the discrepancy between test results and the settlement of the proposed structure by introducing into his arguments one more element of uncertainty, the writer takes it for granted that this discrepancy (together with the possible effects of the lack of uniformity of the underground) represents inevitable defects of the load test results. Hence, when attempting to evaluate the probable settlement of a proposed structure, he bases his judgment not only on test results and his knowledge of soil physics, but also on past experience and on what the test borings show about the nature of the deeper strata. Test results and physical knowledge

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 659.
† *Loc. cit.*, p. 660.
‡ *Loc. cit.*, p. 663.

are used merely for establishing connection between past experience and the new situation. In no case would he dare to rely solely on a test result.

Mr. Williams states* that the writer neglected the effect of the deflection of the foundation on the pressure distribution over the base of the foundation. For foundations on well compacted sand, the pressure-relieving effect of the deflections is very important indeed, but for soft soils, this is insignificant. This fact has been known for more than forty years. Since most raft foundations are supported by soft soils, designers do not often have an opportunity to take advantage of the pressure-relieving effect of deflection. The question concerning the distribution of pressures over the base of raft foundations is still in a controversial state, and the discussion by Mr. White† of the same topic seems to indicate that engineers are by no means unanimous in assuming uniform pressure distribution to be safe.

Dr. Chatley calls attention‡ to the slowness with which hydraulic fills, consisting of fine alluvium, expel their excess water. During 1928, under the writer's supervision, Glennon Gilboy, Jun. Am. Soc. C. E., made an exhaustive study of the process of consolidation of an hydraulic-fill dam. For this purpose, a shaft was sunk through the core of the dam and undisturbed samples were secured at various depths below the crest. The results of these investigations represent an important contribution toward present knowledge of the consolidation process; they conform with Dr. Chatley's observations and with the conclusions previously drawn by the writer from theory.

Dr. Chatley's conception§ of the part that the molecular bond plays in the physics of granular masses is fundamentally sound, although it is still doubtful whether the molecules of two adjoining particles interact directly or whether the interaction merely takes place between the molecules of the skins of adsorped materials surrounding the particles.‖ The possibility undoubtedly exists that the degree of dispersion of a highly colloidal soil may gradually change. To date, very little is known about this process. The writer is inclined to believe that it is often associated with chemical changes and that it plays a very important part in Nature.

The most promising methods for the design of sheet-piling seem to be those based on the assumption of a circular sliding surface. These methods were developed by K. E. Petterson, Sven Hultin, and W. Fellenius, in Sweden.¶

In 1920 the writer was at least as enthusiastic about colloids and about the possible benefits to be derived from colloidal chemistry for foundation and earth-work engineering as Mr. Holmes** is to-day. However, during the succeeding years of struggle with the complex material called soil, this enthusiasm has dwindled to practically nothing. Some of the reasons of the progressive

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 663.

† *Loc. cit.*, August, 1928, Papers and Discussions, p. 1822.

‡ *Loc. cit.*, February, 1928, Papers and Discussions, p. 664.

§ *Loc. cit.*, p. 665.

‖ "Mechanics of Adsorption and the Swelling of Gels," by Charles Terzaghi, M. Am. Soc. C. E., Fourth National Colloid Symposium, 1926.

¶ "Erdstatische Aufgaben," W. Fellenius, Berlin, 1927; "Erddruck, Erdwiderstand und Tragfähigkeit des Baugrundes," H. Krey, Berlin, 1926.

** *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 872.

disappointment can be explained by means of Mr. Holmes' own data. He admits* that "extracting the colloids from a soil to determine the amount of colloidal material is a long and laborious process". Therefore, he proposes a simplified method of his own as shown in Fig. 20.* According to this diagram, an ignition loss of 3% may mean anything between 6% and 29% of colloid content. From the colloid content, Mr. Holmes proposes to derive the "permeability rate". There is no such thing as a definite permeability rate for any soil, particularly for one that contains colloids.

The following data from tests recently performed by the writer on a soil in Spanish Honduras may serve as an illustration. The soil had a clay content of 70% and, in its natural state, a water content of 33 per cent. The coefficient of permeability of this soil was found to be: (*a*) Undisturbed sample under primeval forest with 33% of water, $k = 2.6$ cm. per min.; (*b*) the same soil dried, powdered, and compacted, $k = 0.0024$ cm. per min.; (*c*) the same soil with the same water content as in Case (*a*), but after completely destroying its original structure by kneading, $k = 0.00000085$ cm. per min.

In addition, by increasing the water content of Sample (*c*), from 20% to 40% the coefficient of permeability was increased from 0.00000010 to 0.00000250. Considering these figures, it is difficult to understand how a knowledge of the colloid content could help in determining the hypothetical "permeability rate" of such a soil. Furthermore, it is well known that the same soil with the same colloid and the same water content has very different properties, depending on the character and the quantity of the adsorbed salts which it contains. Nothing except a refined chemical analysis could disclose the nature of these admixtures. Finally, some of the most excellent dam construction materials that the writer has ever encountered, which were splendidly fit for constructing earth dams without a core-wall, contained almost no colloids.

Considering these and many other facts, the writer cannot be blamed for his skeptical attitude toward those who hope to derive important practical benefits from colloid research. He prefers to get his data directly from the results of appropriate experiments. Nevertheless, any advance in the field of colloid chemistry is of outstanding interest to every one dealing with soils and with earthwork engineering. This interest, however, is at present of a purely scientific nature.

Mr. Feld interprets the writer's desire to establish a more accurate interpretation of actual construction experience as a desire to "suddenly substitute a new 'science of foundations'" for precedent and experience.† No such intention ever entered the writer's mind. The need is, not for a latent state of war, but a more intimate co-operation between field and laboratory. In establishing such co-operation, the absence of any authoritative nomenclature certainly is a serious handicap; but it is by no means the principal stumbling block. Far more detrimental is the widespread lack of training in abstract reasoning and the tendency to reject theory completely unless it leads, overnight, to conclusive results.

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 874.
† *Loc. cit.*, p. 877.

The reason the writer neglected to deal with the effect of the shape of the area on settlement* is obvious. At the time when the paper was written, he did not know anything about it. Even in the case of the bearing of solid against solid within the elastic limits, the shape of the area has a very important effect, but all efforts made to solve the problem theoretically thus far have failed. However, since January, 1927, experiments have been conducted at the Massachusetts Institute of Technology, for the purpose of getting at least preliminary information on this controversial subject. The results obtained to date are interesting enough. They suggest that the effect of the shape of the area on both the equivalent settlement and on the ultimate bearing capacity may be very different, depending on the degree of cohesion.

For example, assume two loaded areas, one with a circular shape, of which the diameter is d, and the area, f; and the other one rectangular with the width, d, and a length several times larger than d, so that the area is F. For a material with dominating cohesion (Plasticine), the equivalent settlement for f was found to be smaller than that for F. On the other hand, the ultimate bearing capacity for f was found to be considerably greater than that for F. For perfectly cohesionless materials, the opposite relation was found to exist. The tests were repeated several times, and the results were remarkably consistent. Here, again, it seems a fact that the influence of the shape of the area may be very different, depending on whether the bearing capacity of the material is essentially due to cohesion or to internal friction.

The "sympathetic" settlement of adjoining footings may have very different causes. In the case illustrated by the writer's Fig. 11,† a fence post located at a distance of about 300 ft. from Building (b), had settled several inches. In this case, the sympathetic settlement was due to the presence of a layer of sand beneath a bed of unconsolidated clay. Another type of sympathetic settlement (settlement of the surface of the ground in the vicinity of a concentrated load) has been treated theoretically by the late Professor A. Foeppl in "Vorlesungen über Technische Mechanik", which also contains the results of the tests performed in order to check the theory.

Mr. Housel presents a series of very interesting and valuable test results.‡ However, from these results, he feels justified in concluding that the statements presented by the writer concerning the relation between the bearing capacity and the diameter of round plates, is false and that they lead to absurd results. A similar opinion is known to be held by many other engineers.§ The writer, therefore, may be justified in examining the arguments presented by Mr. Housel in somewhat greater detail.

At the outset of this closing discussion, the writer quoted the assumptions on which Fig. 50 was based. It also was shown that Laws (A) and (B), derived from these assumptions, are as unquestionable as the laws of applied mechanics, provided the assumptions agree with conditions existing in Nature. Therefore, if a discrepancy is found between theory and practice, it must be due to some factor not taken care of by theory. In the case of foundations

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 877.
† *Loc. cit.*, November, 1927, Papers and Discussions, p. 2285.
‡ *Loc. cit.*, April, 1928, Papers and Discussions, p. 1243.
§ *Engineering News-Record*, March 29, 1928, p. 520.

resting on clay, an apparent discrepancy was found to be caused by the low permeability of the material which retards the settlement of a large footing much more than that of a small bearing-block. The advantages to be derived from such knowledge are manifold.

The first lesson learned is the intimate relationship that exists between the low permeability of the underground and the progressive settlement of the buildings which it supports. Furthermore, it is important to understand that a settlement of identical foundations on identical clays may be very different, depending on whether or not the clay contains streaks or layers of permeable sand through which part of the excess water of the clay may freely drain. It is also important to prevent the mistake of applying to a fairly permeable system of strata the experience previously obtained with foundations on very slightly permeable materials, such as plastic, homogeneous clays. That is what the writer calls a "scientific approach" to a soil problem; an attempt to uncover the responsible factors by isolating the variables and systematically determining their relative importance.

However, Mr. Housel selected another and apparently simpler method of dealing with his subject. Instead of attempting to investigate whether or not the statements quoted by the writer were consistent with the accepted laws of mechanics, he introduced into his discussion rather vaguely defined conceptions such as "shearing resistance" per unit per length of perimeter and "pressure bulb".* These conceptions certainly are convenient for a general description of the conditions which exist in the underground of a loaded area; but, if it comes to rigorous analysis, they cease to exist. In applied mechanics, the term, "shearing resistance", refers exclusively to a unit of area, not to a unit of length, and the pressure bulb is not referred to at all, except inasmuch as it indicates vaguely what is vigorously expressed by the formulas of Hertz and Boussinesq.

Considering these facts, it is very difficult to translate into terms of applied mechanics such statements as "the important consideration * * * is the effectiveness with which the different shaped plates adjust themselves to the pressure bulb."†

Speaking of his test results, Mr. Housel admits that the results agree with conditions as defined by Curves C and C_1, Fig. 50. Yet, he rejects them because "* * * when the principle is applied to the much larger areas used in practice * * * the fallacy of this assumption becomes apparent". Then, without preceding analysis, Mr. Housel supersedes the term he rejects by a term of his own conception (Equation (11)),‡ and simply claims that his term represents a satisfactory solution, valid for all soils irrespective of their physical properties. Nowhere in his discussion does he hint at the possibility that a difference in drainage conditions may have a far greater influence on settlements than a difference in compressibility or in internal friction.

Mr. Bailey§ touches the human side of foundation engineering by quoting two cases of failure which occurred in spite of thorough preliminary work.

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1250.
† *Loc. cit.*, p. 1245.
‡ *Loc. cit.*, p. 1249.
§ *Loc. cit.*, p. 1251.

The question arises: Will it ever be possible to avoid entirely such tragic incidents? The writer does not believe so. At the same time, from his personal experience with foundation failures, he is convinced that more than 90% of the accidents which occur nowadays could be avoided by proper application of the results of soil research to the design of foundations. The others will remain inevitable, very likely forever, on account of the fragmentary character of the information furnished by test borings. By transforming this fragmentary information into complete geological profiles, the engineer automatically introduces into his forecast a personal element and with it an element of uncertainty. Thus, on construction work with which the writer was connected, careful test borings were made and all of them indicated the presence of a continuous bed of very stiff clay extending to a considerable depth below the base of the proposed raft foundation. Immediately before the reinforcement for the concrete slab was placed, water broke through the bottom of the pit, carrying large quantities of very fine sand. Subsequent investigation by supplementary boring showed that there was a narrow belt of quicksand enclosed within the clay bed at a very shallow depth below the bottom of the excavation pit and by mere chance the test borings missed it.

The risk caused by the fragmentary character of test-boring records will always continue to exist. In another discussion*, the writer has pointed out that in connection with dam foundations this inevitable risk is still very much greater and very much more serious. The ultimate aim of the science of foundations merely consists in reducing the present intolerably wide margin between the factor of safety of the most and of the least solid foundations.

Mr. Condron† describes an instructive case of foundation experience in connection with the Chicago blue clay. According to the test borings, the stiffness or the cohesion of this clay increased considerably with the depth. Since the cohesion of the clay varied with the depth, and since, in addition, the clay represents a very slightly permeable material, Law (A) is not directly applicable to Mr. Condron's case, and the interpretation of the test results represents a rather complicated problem of soil physics.

Mr. Condron's method of handling the problem can be considered a typical example of a very conscientious and conservative approach along the lines of current practice. In order to determine wherein this practice may fall short of what is desirable, engineers may ask the question: What arguments led Mr. Condron to conclude that 3 000 lb. per sq. ft.‡ is a safe load for the soil which he tested?

His most important argument undoubtedly was that the neighboring buildings, with foundations designed for a soil pressure of 4 000 lb. per sq. ft., showed no evidence of unequal settlement. After Mr. Condron had verified this fact, he made a loading test, with the soil around the loaded area remaining in place. From the favorable outcome of this test, plus the evidence furnished by the neighboring buildings, he reached his conclusion.

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, pp. 1953–1957.
† *Loc. cit.*, May, 1928, Papers and Discussions, p. 1561.
‡ *Loc. cit.*, p. 1563.

If the procedure for determining the admissible soil pressure is so simple, then why does the engineer need any of the more refined soil studies? To answer this rather vital question, suppose that an engineer has to construct a duplicate of Mr. Condron's building. Before preparing the plans for the foundation, he makes exactly the same test as Mr. Condron did, except to assume that there are no neighboring buildings the behavior of which would inform him about the quality of the underground. Furthermore, when making his test borings, suppose that the engineer discovers that, to a depth of 10 ft. below the base of the proposed footings, the underground is the same as that found by Mr. Condron. Below that depth conditions differ in one of three possible ways, as follows:

(*a*) Instead of homogeneous stiff blue clay, there is a homogeneous soft blue clay.

(*b*) Instead of homogeneous stiff blue clay, there is a stiff blue clay which contains several layers of dense but very permeable, water-bearing sand.

(*c*) Instead of homogeneous stiff blue clay, there is a "varved" clay consisting of a succession of almost impermeable and fairly permeable layers about 1 in. thick.

Since, at a depth of 10 ft. below its base, the soil pressure exerted by the loaded 6-ft. square test footing is already negligible, the outcome of the loading test will be almost exactly identical with that obtained by Mr. Condron. Yet, the major part of the pressure bulbs of the full-sized footings will be in a material altogether different from those in the case cited by Mr. Condron. As a consequence, the ratio between the settlement of the test footing and of the full-sized footings will also be altogether different.

In Case (*a*), the settlement due to lateral yield of the soft clay will be much more important than it was when the deeper part of the stratum consisted of stiff material. In Case (*b*), the excess water in the clay beneath the full-sized footings can freely drain into the sand out of the clay, while through the homogeneous stiff clay beneath Mr. Condron's building, it could not percolate except very slowly. In Case (*c*), the clay can discharge its excess water still more freely, and the settlements due to gradual consolidation are apt to be very important although the material gives the impression of being very stiff. Whenever, in practice, the writer encounters "varved clay", he knows there is trouble ahead, and experience has invariably confirmed his anticipation.

The engineer who pays little or no attention to the manifold characteristics of soils would be apt to trust the outcome of the loading test, and would consider Cases (*a*) to (*c*) to be identical with the one described by Mr. Condron. Hence, he would also consider 3 000 lb. per sq. ft. a safe load and to his surprise the result of his decision would be an excessive settlement of the structure. The task of developing a "science of foundations" consists in preventing such errors by painstaking analysis of the relative weight of each individual factor which could possibly influence the behavior of the proposed structure. Without such analysis, the profession would have always as many different

rules for the interpretation of the results of loading tests as there are engineers who keep a record of their observations.

Mr. White's experience, derived mostly from construction work in New York, covers some of the most difficult fields of foundation engineering and, whenever he chooses to disagree with any of the writer's statements, the writer always feels compelled to "think it over twice" before he replies. The two points wherein Mr. White's opinions seem to differ from those of the writer are the value of loading tests and the relation between the bearing capacity and the density of soils.*

The principal arguments against placing too much confidence in loading tests were explained in general at the beginning of this closing discussion and by means of an example in the writer's comment on Mr. Condron's discussion. The writer uses loading tests merely as a means of investigating the character of individual strata, but he never would dare to pass judgment on the probable settlements of a full-sized structure without careful consideration not only of the results of the loading tests but also of the influence of the characteristics of those strata between the elevation of the base of the pressure bulb of the test footing and that of the full-sized footing. The cases in which the soil is of a perfectly uniform character to a depth at least equal to the width of the building are very rare.

The relative density of a soil or of a rock is undoubtedly a factor of importance, but unfortunately the relation between the behavior of this soil and its density are by no means simple enough to furnish reliable information on the bearing capacity of the material. Thus, when experimenting with clays, the writer had an opportunity to compare two blue clays from two different localities. One of these clays, with a water content of 35% (volume of voids, 48%) was almost liquid, while the other with exactly the same water content and the same volume of voids was found to be stiff and had a very appreciable consistency. Similar although less conspicuous variations were found to exist when experimenting with sands.

This leads to a discussion of the effect of the shape of the grains. Mr. White† is inclined to assume that the large voids ratio may have the same effect on the mechanical properties of a sand as the mica content. As a matter of fact, the compressibility of the same sand increases rapidly with its volume of voids. There is, however, one fundamental difference between pure sand and sand-mica mixtures which cannot be explained by Mr. White's assumption. This difference concerns the elasticity of the two materials as illustrated by the shape of the rebound curves. Thus, when experimenting with very fine quartz dust, it was found that the compression curve of this material was almost as steep as that of a sand-mica mixture, but the rebound curve was almost as flat as that of an ordinary sand, while the rebound curves of the equivalent sand-mica mixture rose very sharply. Since the rebound curves of the clays are as steep as those of sand-mica mixtures, the writer concluded that the clays and silts owe their mechanical properties essentially to the presence in the material of a high percentage of scale-like particles. At the

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1831.

† *Loc. cit.*, p. 1832.

time when this statement was published, it was presented as a hypothesis. Since then, Professor Goldschmidt, in Oslo, Norway, has succeeded in measuring the actual quantity of scale-like particles present in typical Norwegian clays,* and the data which he obtained checked so well with those derived from the sand-mica experiments performed at Massachusetts Institute of Technology that there is little doubt about the hypothesis being fundamentally sound.

From a practical point of view, the principal difference between very fine sand and an equally fine sand-mica mixture is that the volume of voids in the former is always very much smaller than that of the latter, provided both were deposited by identical geological processes. Sand-mica mixtures with macroscopic grains and with a high percentage of mica have not yet been found in Nature. The so-called "micaceous sands" as a rule contain only a very small fraction of mica by weight. The micaceous appearance is due to the fact that a single sand grain weighs twenty or more times that of a mica flake of equal size.

Mr. White† is inclined to doubt the validity of the writer's statements concerning the effect of the depth of a foundation on the settlement for perfectly cohesionless material. His doubts are unquestionably based on what he has observed in actual practice; and, in actual practice, he has almost no chance to meet a material conforming with the condition of perfect lack of cohesion.

Mr. White's examples of the general character of the settlement of whole buildings (Fig. 31)‡ leave little doubt that the centers of the outside walls settled more than the corners, but it is still an open question whether the centers of these buildings settled more than their perimeters. In order to insure uniform settlements, some engineers in Texas follow the practice of keeping the soil pressures for the outer footings 12 to 20% less than those for the inner footings, and their experience seems to confirm the soundness of their practice. It should also be considered that the relation between the amount of settlement of the center and of the perimeter of a building, at equal soil pressure per unit of area may be very different, depending on the degree to which the cohesion of the underground dominates over internal friction.

The introductory remarks of Mr. Simham's discussion§ describe exactly what the writer has been "driving at" for many years. The writer also concedes that no laws can be derived from uncorrelated sets of experiments; but, at the same time, he seriously doubts whether any formula can be derived to cover all the requirements of actual practice. In dealing with foundation problems, the writer confines himself to what Mr. Simham calls "a careful diagnosis of every factor that influences the behavior of the soil", and in doing so he has almost invariably found that he must be satisfied with finding out to what extent his previous experience applies to new cases. All the rules which he derived, and very likely all those which he will derive in the future, are merely intended to be tools for use in this fundamentally important diagnosis.

* Abstract by Charles Terzaghi, M. Am. Soc. C. E., in "Economic Geology," 1928.
† *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1833.
‡ *Loc. cit.*, p. 1828.
§ *Loc. cit.*, p. 1834.

Equations (12) to (21),* inclusive, represent one possible approach among many to a mathematical description of what engineers may observe. However, their value will depend on the degree to which Mr. Simham succeeds in taking care of the time factor, which is the stumbling block in the path of all similar attempts.

Mr. Marston† describes an example for the application of soil physics to the study of materials for hydraulic-fill dams. Since this investigation was terminated, much progress has been made along similar lines by the work performed in the writer's laboratory. In pursuing this work, the thorough investigation of perfectly undisturbed materials extracted from the core of an existing hydraulic-fill dam proved to be of inestimable assistance and threw much light on the disputed problems. (See, also, the writer's comments on Dr. Chatley's discussion).

Mr. Shaw‡ describes interesting experiences with New Orleans muck soils. Since the internal friction of under-drained muck soils is extremely low, their bearing capacity depends essentially on their cohesion. At the same time, that they are fairly permeable is proved by the fact that it is possible to drain them with open ditches. For soils of such a character the rule represented by Curves C and C_1, Fig. 50, has at least approximate validity. This statement agrees with Mr. Shaw's experiences.

Furthermore, previously published theoretical investigations of the writer have led to the following conclusions regarding the ultimate bearing capacity of medium permeable and saturated silts and muck soils. Select on the surface of such a soil two equal areas and load both of them at a different time rate until failure occurs by an "out-and-up" flow of the loaded soil. The more slowly the load is applied, the more chance the soil has to expel its excess water during load application. Therefore, ultimate failure of the area which was loaded very slowly will occur at a higher load per unit of area than that on which the load was rapidly applied. This theoretical conclusion is substantiated by the success of Mr. Shaw's method of levee construction by successive application of wide and thin layers. The compacting effect produced by load application is illustrated by the writer's Fig. 4.§

Mr. Shaw's observations concerning the difference between the effect on soft soils of concentrated and of distributed loads call attention to an important fact, closely related to the influence of the depth of foundation on the ultimate bearing capacity of the soil. Suppose the area, a-a (Fig. 51), is uniformly loaded, while on both sides of a-a, the load decreases from its value at a toward zero. For this case, both theory and experience lead to the conclusion that the wider the area over which the load spreads, the greater is the load p, p_1, p_2, etc., at which failure occurs.

The effect of the flaring out of the load becomes still more conspicuous if the thickness of the soft layer is very much smaller than the width of the base of the fill (Fig. 52). In this case, the soft layer is "trapped" between the base of

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, pp. 1835–1837.
† *Loc. cit.*, p. 1838.
‡ *Loc. cit.*, October, 1928, Papers and Discussions, p. 2313.
§ *Loc. cit.*, November, 1927, Papers and Discussions, p. 2272.

the load and the surface of the stiff layer, whereby an arching effect, similar to that which takes place in grain bins, prevents the soft layer from lateral flow. Very striking examples of the effect of the ratio between the width of fills and the depth of the loaded layer of soft material on the ultimate bearing capacity of the soil were observed by Mr. V. R. Burton.* From his experiences with the construction of fills on peat deposits in Michigan, Mr. Burton arrived at the conclusion that in placing a fill with a width ranging between 24 and 30 ft. on the surface of a peat deposit, the character of the resulting subsidence was very different, for varying thicknesses of the deposit. If the thickness was less than about 20 ft., the fill merely produced a trough-like depression in the peat deposit. On the other hand, if the thickness of the deposit exceeded about 26 ft., the fills penetrated as far as the bottom of the deposit, thus completely displacing the peat beneath the center line of the fill. Examples of the sudden subsidence of fills placed on soft muck soils can be found in the report of the Swedish Geo-Technical Commission.† The observations made by the Swedish engineers in connection with large-scale subsidence failures led them to base their stability computations for such soils on the assumption of circular sliding curves.

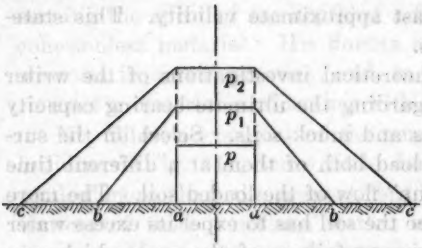

FIG. 51.—INCREASE IN BEARING CAPACITY OF UNDERGROUND DUE TO WIDENING FLARE OF TRAPEZOIDAL LOAD.

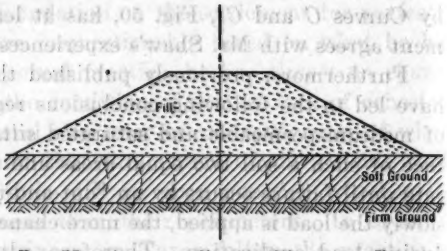

FIG. 52.—ARCHING EFFECT WITHIN A THIN, SOFT LAYER LOCATED BETWEEN BOTTOM OF FILL AND TOP SURFACE OF FIRM GROUND.

Mr. Shaw's experiences furnish instructive examples of the fact that the results obtained by theory can be depended upon, provided the assumptions conform reasonably well with the conditions existing in Nature.

It was a great pleasure to survey the valuable test results presented by Mr. Wing.‡ In regard to the loading tests the writer would consider the tests finished as soon as the settlement becomes equal to about one-fifth the diameter of the loaded area. Beyond this limit the process of settlement passes into one of penetration. When making the tests oil-jacks are always preferable to screw-jacks.

The writer uses as a criterion for the bearing quality of sand deposits the settlement under a load of 500 lb. per sq. ft., provided the loaded area is at least equal to 1 sq. ft. Under such a unit load, a well-compacted sand deposit does not settle more than about 0.001 ft. The loosest deposit that ever came

* "Fill Settlements in Peat Marshes," by V. R. Burton, *Proceedings*, Sixth Annual Meeting, Highway Research Board, 1926, pp. 93–113.

† Statens Jarnvägars Geotekniska Comm., 1914–22. An English abstract and comments on the report by Charles Terzaghi, M. Am. Soc. C. E., are filed in the Carnegie Library, Pittsburgh, Pa.

‡ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 83.

under the writer's observation settled under the same load through a distance of ¼ in. From the character of the settlement curves for the sand (Fig. 44),* the writer suspects that the sand was loosened by the upward current of water, as the pit was being pumped dry. On account of this possibility, the writer keeps the water out of a test pit by pumping, uncovers two opposite vertical sides of the pit, and makes the loading test in a horizontal direction, by inserting a hydraulic jack between two bearing plates, applied to the sides of the pit. The bearing plates should have a clearance, all around the rim, equal to at least twice the diameter of the plate. Experience has shown that the results of such tests differ but little from those made on horizontal plates, and one is sure that the loaded material is in an undisturbed state.

The results of mechanical analysis should preferably be presented in graphical form with the grain size plotted on a logarithmic scale.† The limits of consistency of the clay, quoted by Mr. Wing, remove any doubt concerning the type of clay that he encountered. Experience has shown that, for natural clay deposits, both the limits of consistency and the water content vary within rather wide limits. Therefore, it is advisable to perform these tests for each drill hole on several samples. Instead of determining the volume change associated with a change of the moisture content, it is preferable to determine the shrinkage limit (from dry weight and from the volume of the dried sample, assuming for the dried substance a specific gravity of 2.7).‡

The settlement diagram (Fig. 45),§ for the clay indicates that the loaded area was too big for the size of the test pit. The abnormal shape of the curve seems essentially to be due to arching action within the clay, between the bearing plate and the walls of the pit. Similar phenomena were observed by the writer on a small scale, when loading clay specimens that were contained in a rigid box. The clay beneath the bearing plate undoubtedly compacted, but this fact alone does not account for the characteristics of the curve. The width of the pit should be equal at least to five times the diameter of the loaded area.

Loading tests on both sands and clays should always be made at least in duplicate on what appears to be the stiffest and the softest spot of the underground. The writer selects these spots after having passed over the ground with a sounding rod. The observations to be performed on the diversion dam‖ are most promising, particularly if the settlement observations extend over a longer period and, if, in addition, bench-marks are established on the unoccupied ground at a distance equal to one-half the width of the structure.

In his very thorough and able discussion,¶ Professor Krynine quotes certain statements made by Dr. Ing. Koegler concerning the relation between settlement and size of area, and finds them to be at variance with those of the writer, as far as soils with great cohesion are concerned. It should be emphasized

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 86.

† "Physical Differences Between Sand and Clay," by Charles Terzaghi, M. Am. Soc. C. E., *Engineering News-Record*, December 3, 1925, Fig. 2.

‡ "Present Status of Subgrade Testing," by C. A. Hogentogler, Charles Terzaghi, and A. M. Wintermayr, *Public Roads*, March, 1928.

§ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 87.

‖ *Loc. cit.*, p. 88.

¶ *Loc. cit.*, November, 1928, Papers and Discussions, p. 2506.

that Dr. Koegler derived his conclusions exclusively from tests performed with cohesionless sand. For this reason they conform to Curves C and C_2, Fig. 50. For the same reason they certainly do not apply to soils in general, irrespective of their physical characteristics, and Dr. Koegler's generalized statements must be received with reserve until more convincing arguments are presented for substantiating their validity.

Professor Krynine's suggestion to consider, in the computation of a raft foundation, the presence in the ground of hard and soft spots is theoretically sound. However, in practice, it seems very difficult to follow the advice. First, loading tests would be required, not only all over the area, but also at different elevations below the ground surface. Furthermore, since engineers do not yet know the distribution of the soil reactions for perfectly homogeneous foundations, and the importance of the effect of the soil characteristics on this pressure distribution, there is little hope for successfully introducing further refinements into computations.

The doubts expressed* regarding the writer's comment on the settlement of the hardpan caissons at the Chicago Union Terminal, were apparently due to the fact that Professor Krynine was not familiar with the details of the tests. The frictional resistance acting over the surface of the caissons was determined by undermining the caissons and loading them until they slipped. The unit loads quoted by the writer obviously represented the top load minus the relieving effect of side friction.

Another objection by Professor Krynine concerns the shape of the curve which represents the distribution of the soil reactions over the base of a loaded slab.† When the writer stated that these curves have a parabolic shape, he obviously meant an approximately parabolic one. At the rudimentary state of present knowledge a final statement concerning this shape would be premature. In addition, the difference between the right and left sides of Fig. 40† is so slight compared with the importance of the errors associated with Professor Krynine's theoretical assumptions that it seems reasonable to disregard it. Very much more important is the question as to whether the parabolic law may not lose its validity altogether if applied to very cohesive soils. For such soils the pressure distribution is apt to be similar to that over the base of a piston which acts on the plane surface of a homogeneous, elastic solid. The writer doubts seriously whether this distribution has any resemblance at all with either one of the distributions represented by Fig. 40.

Professor Krynine's comments‡ on Fig. 5 are fully justified. The writer agrees that the two lines will be neither straight nor parallel. The reason he failed to show their true shape in the diagram was simple enough. He does not yet know this shape.

The writer became interested in the subject of "entrapped air" for the first time when certain inconsistencies were found to exist between the results of compression tests on clays that were dried before testing and on others that were used as they were taken from the drill hole. The inconsistencies were

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2506.
† *Loc. cit.*, p. 2508.
‡ *Loc. cit.*, p. 2509.

caused by the fact that those samples which were prepared by mixing the dried specimen with water contained an appreciable quantity of air, while the others contained practically none. In order to determine the importance of the influence of the air on the test results, Mr. A. Casagrande performed some tests in the writer's laboratory. Fig. 53 shows a typical test result. One of the two samples merely contained air and water. The other was prepared from dry powder and, in addition, air was injected into the specimen by a drop counter in such quantities that its voids ratio increased by almost 50 per cent.

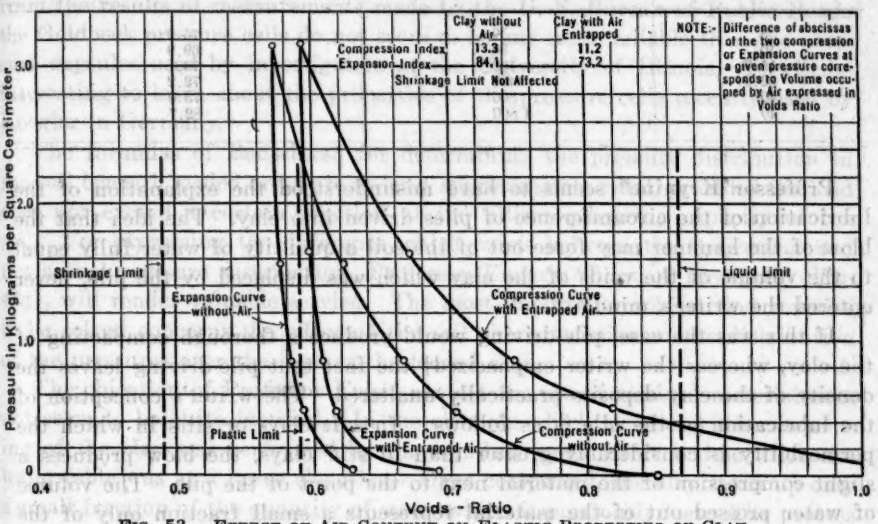

FIG. 53.—EFFECT OF AIR CONTENT ON ELASTIC PROPERTIES OF CLAY.

The most conspicuous effect of the air content was the difference in shape of the time-compression curves of the samples. This is caused by the fact that the compression of the air takes place almost instantaneously similar to the compression under pressure of the air bubble in a spirit-level, while the compression of the clay occurs gradually, according to the laws of consolidation of compressible capillary systems. By this very reliable criterium, and by the results of numerous air-content determinations, the writer has learned that neither clays nor silts seem to contain in Nature a measurable amount of air, provided they are beneath the zone of temporary desiccation. For this reason the writer did not consider it necessary to mention the air factor in connection with a paper on foundations. It plays an important part merely in connection with sub-grade studies for roads. The intermittent wetting of the sub-grade of hard surface roads occurs almost entirely by capillary action, whereby a considerable quantity of air remains in the soil.

Some experiments on the air content of fine sands and dust, wetted by capillary action, were conducted in 1926 by Mr. C. V. Arellano, under the supervision of the writer.* The test results seemed to indicate that the air content decreases considerably with time. One of the samples tested was a fine, reddish, micaceous dust; effective size, 0.028; uniformity coefficient, 3.6;

* "Relation between Permeability of Sands and Capillary Upward Movement of Water," by C. V. Arellano, Master's Thesis, Mass. Inst. Tech., 1926.

specific gravity of grains, 2.674; voids ratio, 0.698; and coefficient of permeability, 9.75×10^{-5} cm. per sec. The results are shown in Table 12.

TABLE 12.—AVERAGE DEGREE OF SATURATION OF FINE DUST WETTED BY CAPILLARY RISE.

Capillary rise, in centimeters.	Time elapsed since water started to rise, in minutes.	Percentage saturation, or the ratio between total volume of water and total volume of voids.
5	9	75.2
10.	38	69.9
15.7	100	70.1
32.9	487	72.2
40	800	73.0
40	3 800	82.7

Professor Krynine* seems to have misunderstood the explanation of the lubrication of the circumference of piles driven into clay. The idea that the blow of the hammer may force out of the soil a quantity of water fully equal to the volume of the voids of the clay which was displaced by the pile, never entered the writer's mind.

If this was the case, pile-driving would produce a thorough compacting of the clay, whereas the writer emphasized† the fact that pile-driving leaves the density of the clay deposits practically unaltered. The writer's conception of the lubrication of the pile is as follows: In soft clays or silts in which the permeability is considerably greater than in stiff clays, the blow produces a slight compression of the material next to the point of the pile. The volume of water pressed out of the material represents a small fraction only of the volume displaced. Lubrication is caused by this small quantity escaping between the soil and the surface of the pile. The absorption of the film during the period between driving and re-driving is due to subsequent expansion of the slightly compressed clay which surrounds the pile. This process of absorption by no means requires (as Professor Krynine assumes) the presence of entrapped air, because it also occurs in the laboratory under test conditions which completely exclude the presence of air in the clay.

Professor Krynine emphasizes‡ the leaning of Russian engineers toward the "dry substance theory". On account of this leaning, Russian engineers seem to be inclined to speak of the "bearing capacity of soils" in general without due regard to their extremely manifold physical characteristics. This, at least, is the impression the writer has obtained from some recent Russian publications on this subject.§ More than a half century ago, a prominent French engineer expressed his experiences with earth slides in a single sentence which may be translated: "The fear of frost and of clay marks the beginning of wisdom". In a similar fashion the writer is inclined to summarize his experiences in dealing with the bearing-capacity problem in the sentence:

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2511.

† *Loc. cit.*, November, 1927, Papers and Discussions, p. 2281.

‡ *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2514.

§ Brief abstract of chapter on "Bearing Capacity," by Mr. N. Ivanoff, in Textbook on Soils and Top Soils, of the Russian Highway Research Bureau, Leningrad, 1926.

"To recognize the full importance of the permeability factor marks the beginning of wisdom in foundation engineering". Since the permeability factor represents by far the most obstinate stumbling block in the path of any attempt to bridge the gap between small-scale tests with clays and actual foundation practice, the writer cannot help regarding "dry theories" of soil behavior with deep distrust. The history of pile formulas should serve as a solemn warning.

Professor Enger's valuable contribution* calls attention to the great difficulty of measuring the stresses that act within the loaded earth. Judging from the results of measurements made by the U. S. Bureau of Public Roads the Goldbeck pressure cells do not seem to be any more reliable than the pressure capsules used by investigators at the University of Illinois. It will be interesting to learn about the properties of the pressure cells recently used by Koegler in Germany.†

The formulas of Boussinesq for determining the pressure distribution in the soil beneath loaded areas are at least approximately valid, provided the load does not exceed a certain fraction of the ultimate bearing capacity of the soil. For expressing the difference which exists between the theoretical and the actual pressure distribution, Professor Enger's "lines of pressure" (Fig. 48‡), will render valuable service. The most important problem that remains to be solved concerns the effect of the soil characteristics on the distribution of the pressures over the base of the loaded area.

The objection of Professor Enger against the use of remixed material does not seem to be quite justified. In the example which he cites,§ the compacting of the clay was performed by the capillary pressure at a time when the water-table was at a lower elevation. Once compacted, the clay takes up only a small fraction of the quantity of water which it previously lost. The process can be reproduced very easily in the laboratory by first compacting the clay under external or capillary pressure (applying the pressure under water or exposing the surface of the clay to the air) and then allowing it to expand very slowly under water.

In Nature one even encounters layers of very stiff clay beneath the bottom of lakes, between layers consisting of the same material in a very soft state. The history of these "fossil crusts" is as follows:‖ At a period when the surface of the older clay deposit was exposed to the air, the top layer of this deposit partly dried out (compression due to capillary pressure). At a later date, the surface of the older deposit was flooded, and a new soft clay deposit was formed on top of it. Due to the very imperfect volume elasticity of the clay, the former crust retained the greater part of its stiff consistency (see, for instance, the re-saturation or swelling curves in Fig. 53).

Since the paper was published, much headway has been made in developing methods for securing undisturbed samples of clay.¶ Tests are being planned

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 91.

† "Ueber Tragfähigkeit von Sandschüttungen," *Bautechnik*, 1927.

‡ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 93.

§ *Loc. cit.*, p. 95.

‖ Statens Jarnvägars Geotekniska Comm., 1914–22, English abstract and comments by Charles Terzaghi, M. Am. Soc. C. E., filed in Carnegie Library, Pittsburgh, Pa.

¶ "Relation Between Permeability of Sands and Capillary Upward Movement of Water," by C. V. Arellano, Master's Thesis, Mass. Inst. Tech., Cambridge, Mass., 1926.

to investigate the relative importance of the disturbance caused by different sampling tools. There seems also to be no difficulty in securing undisturbed clay samples from depths as great as 100 ft.

The effect of vibrations on the settlement of structures resting on clay was brought to the writer's attention under rather peculiar circumstances. Loading tests were made on a small scale on glacial clay, in his laboratory, and very sensitive instruments were used for measuring the settlements. All the settlement curves showed distinct breaks at points corresponding to 5:00 P. M. and 9:00 A. M., the 9:00 A. M. to 5:00 P. M. sections being considerably steeper than the 5:00 P. M. to 9:00 A. M. sections. The cause of this phenomenon was found to be that between 9:00 A. M. and 5:00 P. M. the different motors located in the Massachusetts Institute of Technology Laboratories are operating, while at 5:00 P. M., they are shut down.

The tests suggested by Mr. Jorgensen* will be particularly valuable if they are made in triplicate, using identical bearing plates, resting on identical clay water mixtures, so that one of the bearing plates consists of cast iron (impermeable), one of concrete (medium permeable), and one of concrete separated from the clay by a sheet of filter paper and a thin layer of sand (permeable). The diameter of the bearing plates should not be more than one-fifth that of the container. The exposed surface of the clay should be covered with a paraffined rubber sheet in order to prevent evaporation, and the settlements should be measured with delicate instruments to detect the difference in the speed of settlement of the three plates. During the tests, the load should be kept constant, and should be equal to the pressure required to produce an initial settlement of the cast-iron bearing plate equal to about 0.05 or 0.1 times its diameter. In each of the three cases the load should be applied in equal increments and in equal time intervals. In order to obtain the maximum benefit from the tests, the base of the bearing plates should be equipped with annular oil-pressure cells, for investigating the distribution of the soil reaction over the case.

The procedure proposed by Mr. Jorgensen for load-moisture tests is practically identical with that of the writer. Since the shape of the re-compression curves, *c d* (Fig. 49†), is only slightly influenced by the intensity of the pressure at which swelling was begun, no great care is necessary in performing the test.

In conclusion, the writer wishes to express his gratitude to the discussors for their valuable contributions. The gaps in the presentation of the subject are still numerous. Yet, the arguments brought forth offered a unique opportunity for expanding the paper on those points which were still in doubt in the minds of the readers.

The results of the discussion may be summarized, as follows:

1.—The *Engineering News Formula* received all the comment it deserves and few engineers seem to be willing to defend it.

2.—The attitude of the different engineers toward the writer's theoretical statements represented by Fig. 50 was far less unanimous, yet it leads to

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 96.
† *Loc. cit.*, p. 97.

very interesting conclusions because no obvious contradiction between theory and practice was found to exist by those who reported experiences with fairly permeable materials (Messrs. White, with friable limestone; Shaw, with muck; and Wing, with sand and gravel). Without exception, all the examples which were quoted as arguments against the validity of the writer's "rules" referred to clays. The reason thereof is obvious. The compression of clay under load proceeds very slowly. Large footings or raft foundations require decades or even hundreds of years for complete settlement, while the observations on which the arguments were based covered, at the very best, a period of a couple of years of the settlement records. In other words, the bill for the excessive loading of a sand or silt foundation should be paid without delay by the first generation, while that for treating the clay in a similar manner should charged against the second or third generation.

It is amazing to realize that the Engineering Profession is facing the problem of progressive subsidence of clay foundations in almost every part of the globe, and yet is still completely ignoring the time element when it comes to approaching the settlement problem from the theoretical point of view. If the discussion had not brought forth anything but striking illustrations of this one fact, the labor involved could be considered as well invested.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE ROLE OF THE CIVIL ENGINEER IN POWER DEVELOPMENT

Discussion*

By I. W. McConnell, M. Am. Soc. C. E.†

I. W. McConnell,‡ M. Am. Soc. C. E. (by letter).§—There is little that the writer can add to the discussion. Colonel Junkersfeld emphasizes‖ the value and imperative necessity of a broad viewpoint. His composite table (Table 16),¶ showing a range of costs under widely varying conditions, gives a "bird's-eye view" of the situation much more compactly than any of the isolated cases presented by the writer. It confirms the opinion that the civil engineer has as much incentive as any other type of engineer to expand his control of facts, economics, and human engineering into the broad field of the administrator and originator of power enterprises.

Mr. Orrok** draws entertainingly, as always, on his great fund of experience gathered over a wide field and in an advantageous position from which to view the whole panorama. He emphasizes, with Colonel Junkersfeld, the dependence of modern economic performance upon well-rounded and balanced use of modern specialties. The evolution of the modern compact, visually unimpressive, but powerful steam turbine unit from the awe-inspiring reciprocating engine used in 1910 is one of the most fascinating accomplishments of an age of wonders. The mechanical accomplishment is as wonderful as the constant encroachment of added efficiencies upon the waste factor.

In the list of economics cited by Mr. Orrok it is a comfort to call attention to improvements in building methods, such as increased spans, increased

* Discussion of the paper by I. W. McConnell, M. Am. Soc. C. E., continued from August, 1928, *Proceedings*.

† Author's closure.

‡ Vice-Pres., Dwight P. Robinson & Co., Inc.; also, United Engrs. & Constructors, Inc., Philadelphia, Pa.

§ Received by the Secretary, February 13, 1929.

‖ *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 923.

¶ *Loc. cit.*, p. 924.

** *Loc. cit.*, p. 925.

heights, lighter and stronger walls, floors, and roofs, which have kept pace with rational requirements for modern machinery.

Messrs. Justin* and Scheidenhelm† contribute pertinent comment as to the manifest preponderance of civil engineering features in hydro-electric power propositions.

The writer hoped the discussion might incite some of the younger minds of the profession to study how to use technical training and thereby turn precise and orderly habits of thinking into stepping stones to those positions of command where final decisions as to great enterprises are made. Lofty ambition is, perhaps, not the predominating quality developed in the plodding tasks of engineering. On the other hand, however, some enterprises would not be injured if they had to respond more faithfully to the common sense scruples which the successful engineer must cultivate. Perhaps a continuity of engineering direction through all the years, would have kept the plans of the mythical De Couvrier in Mr. Sargent's illustration,‡ constantly in the "tickler" file, to the end that unnecessary and misleading reports of investigators less competent would not have brought about actual construction and use of less worthy property.

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* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 928.
† *Loc. cit.*, p. 929.
‡ *Loc. cit.*, August, 1928, Papers and Discussions, p. 1918.

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PAPERS AND DISCUSSIONS

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THE COMPRESSIBILITY OF SAND-MICA MIXTURES

Discussion*

BY GLENNON GILBOY, JUN. AM. SOC. C. E.†

GLENNON GILBOY,‡ JUN. AM. SOC. C. E. (by letter).§—Mr. Simham's mathematical analysis‖ of the characteristics of mixed sediments is very ingenious, and the results are in remarkable accord with the facts. The writer made further computations on the basis of Mr. Simham's assumptions, and found that the computed changes in voids-ratio under a certain assumed change in load were reasonably close to the actual values. For the actual case, the curve of change in voids-ratio under a given pressure, plotted against mica content, lies below Mr. Simham's theoretical line in the lower region, where the sand predominates, and above in the upper region, where the mica is the principal factor. The point of crossing is at about 40% for the mixtures used in the tests. These facts indicate that Mr. Simham's assumption of independent action of the constituent parts is not strictly valid. Its accuracy can be determined only by an elaborate series of experiments.

It is the writer's opinion that Mr. Simham's computations will be found of practical value only for estimating the magnitude of the quantities involved before performing tests on artificial mixtures in the laboratory. If it were possible to perform, on a natural soil, an accurate mechanical analysis which would result in the determination of the shape as well as the size of the grains, it is conceivable that the compressibility characteristics might be computed from such an analysis by an extension of Mr. Simham's method; but it is much simpler and more accurate to measure the compressibility of the soil directly.

Mr. Rubey¶ attacks the problem from an unusual angle, that of specific internal surface. The writer is not in agreement with this theory, for two reasons: First, it is difficult to conceive of the existence of "cushion-like films" of sufficient thickness to cause the high compressibilities observed in the tests; and, second, Mr. Rubey's hypothesis necessarily involves the conclusion

* Discussion of the paper by Glennon Gilboy, Jun. Am. Soc. C. E., continued from November, 1928, *Proceedings*.

† Author's closure.

‡ Research Associate, Dept. of Civ. Eng., Mass. Inst. Tech., Cambridge, Mass.

§ Received by the Secretary, February 25, 1929.

‖ *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1933.

¶ *Loc. cit.*, p. 1936.

that sands with grains of different size will have widely differing voids-ratios and widely differing compressibilities. In other words, if Sand A is composed of grains 0.1 mm. in diameter and Sand B of grains 1 mm. in diameter, Sand A should have a voids-ratio, in a given state of compactness, ten times that of Sand B in the same state, and ten times the compressibility, since Sand A has ten times the specific surface of Sand B. This is obviously untrue.

Mr. Rubey's straight-line relations between specific surface and volume cannot be lightly dismissed. (See Fig. 5.)* The writer is of the opinion, however, that this is one of the instances commonly encountered in which an apparently obvious relationship between two quantities does not really exist. This statement is difficult to prove by direct reference to a soil mass. However, the writer has already alluded to his opinion that the mica flakes bend like tiny beams, so that an analogy based on the ordinary beam theory may serve to illustrate the idea.

Consider a series of simple rectangular beams, of varying length but of similar proportions; that is, length, l; depth, $d\,l$; and breadth, $b\,l$; l being variable, while b and d are constant. Assume all the beams to have the same modulus of elasticity and to be loaded in the same manner with the same load, W.

The maximum deflection of any beam of this series will be $C\frac{W\,l^3}{E\,I}$, in which, C is a constant that depends on the load distribution. The value of I is $\frac{(b\,l)\,(d\,l)^3}{12}$.

Grouping all the constants into a single constant, K_1, the deflection is, $\frac{K_1}{l}$. The surface per unit volume of any one of the beams is,

$$\frac{2\,(b\,l)\,(d\,l) + 2\,(b\,l + d\,l)\,l}{(b\,l)\,(d\,l)\,l} = \frac{K_2}{l}$$

in which, K_2 is a constant.

This analysis shows that both deflection and specific surface of the series of beams under consideration are inversely proportional to length. A graph of deflection against specific surface, therefore, will be a straight line; but one would scarcely be justified in the conclusion that the deflection of a beam depends on its specific surface.

Mr. Feld† is quite justified in surmising that experiments on minerals other than those actually used would probably give somewhat different results. The writer was merely endeavoring to show the general nature of the effect of scale-like particles, and was not especially interested in detailed variations.

The writer regrets that he gave the impression of attempting to deduce relative settlements of buildings on the basis of mica content alone. The intention was to illustrate in a simple manner the probability of making serious errors in settlement calculations based on indirect evidence and unsupported by direct physical tests of the soil properties which govern the rate and amount of settlement.

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* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1937.

† *Loc. cit.*, October, 1928, Papers and Discussions, p. 2357.

Mr. Feld claims* that the writer "has erred somewhat in stating that particles of colloidal size do not have properties different from materials containing particles of larger size * * *". If this statement had been made, it would have been an error; but no such statement appears in the paper.

Mr. Feld also states* that "the author has erred * * * in the statement that the scale-like grains should be accorded a major consideration in ideas of soil behavior". The experiments show very definitely that the presence or absence of mica has an enormous influence on the compressibility of a soil; and the writer believes that any factor which has such a marked effect on such an important property is necessarily entitled to careful consideration.

The writer is of the opinion that Professor Krynine's statement† that "there is no analogy between soil and other engineering materials", is somewhat too strong. It is true that soils behave in a manner different from other construction materials; but it is also true that many of the theories applicable to solid bodies, such as the theory of rupture, stress distribution, etc., are to a great extent applicable to soils as well. The time element, in particular, is not at all peculiar to soils. Fatigue in metals and plastic flow in concrete are only two of the many dynamic phenomena which are of great importance in the study of materials of engineering.

The writer agrees with Professor Krynine that there is danger of error in applying too rigidly the results of experiments with macroscopic particles to materials consisting of microscopic grains. Such a rigid application was not intended to follow from these researches. On the other hand, Professor Krynine's statement‡ that "the experiments seem to have failed in attaining their object", is not correct. The object, it is true, was very modest, namely, to demonstrate that the presence or absence of mica in a soil has a marked effect on its compressibility, elasticity, and density. That object has been attained.

Since the researches were terminated, three items of corroborative evidence have come to the attention of the writer: (1) The investigations of Messrs. Ross and Shannon§ which demonstrate that Bentonite, one of the most compressible clays known, is predominantly micaceous in character; (2) Professor Krynine's simple experiment on capillary rise‖ which showed indirectly that clay particles are scale-like; and (3) the results reported by Professor V. M. Goldschmidt¶ of detailed investigations on a great many Norwegian clays, showing that a large proportion of the particles comprising these natural clays are scale-like, particularly in the finer fractions.

The writer feels that these results tend to add weight to his original contention that the presence of flat particles is chiefly responsible for the high voids-ratios and high compressibility of fine-grained soils.

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* *Proceedings*, Am. Soc. C. E., October, 1928, Papers and Discussions, p. 2358.

† *Loc. cit.*, November, 1928, Papers and Discussions, p. 2543.

‡ *Loc. cit.*, p. 2544.

§ "The Minerals of Bentonite and Related Clays and Their Physical Properties," *Journal*, Ceramic Soc., Vol. 9, 1926.

‖ "Elementary Proof of Shale-Likeness of Clay Particles," *Public Roads*, January, 1928.

¶ "Undersøkelser över Lersedimenter," Beretning om Nordiske Jordbrugsforskeres Kongres i Oslo, 1926. (A review of this paper by Charles Terzaghi, M. Am. Soc. C. E., appeared in *Economic Geology*, June–July, 1928.)

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CONTINUOUS BEAMS OVER THREE SPANS

Discussion*

By I. Oesterblom, M. Am. Soc. C. E.†

I. Oesterblom,‡ M. Am. Soc. C. E. (by letter).§—The response to this paper has been very gratifying; that numerous discussions have been submitted for an extended period would possibly indicate that the question raised has real and timely interest. Generally this has been admitted by the discussors. The purpose and also the contents of the paper have been commented on with approval, and whatever criticism has been submitted has been searching and constructive. Some of it has gone beyond the scope of the paper, but it has been valuable, nevertheless. Thus, from two different impulses, there have been secured supplementary, useful ideas and methods as well as definite information in regard to simplified designs of continuous beams.

Some of the critics have definitely expressed the opinion that a great deal was omitted from the original paper. The question might be raised, however, if a very complete paper on the subject would not have been an unfortunate contribution. It would have reduced the opportunity of supplementary contributions from interested engineers, but even worse than that, it would have entered into the more general field of stresses and strains in elastic frames, already so thoroughly and competently treated by a legion of highly qualified authorities.

It was the purpose of the paper—unfortunately it seems to have been insufficiently emphasized—to reduce to easily workable form some of the very complicated combinations of formulas for continuous beams, so that reasonably correct moment factors might be quickly ascertained for the most common cases of simple continuity.

Perhaps the examples given were presented in a manner which seemed to contradict the purpose of speed in execution. By neglecting variability of

* Discussion on the paper by I. Oesterblom, M. Am. Soc. C. E., continued from February, 1929, *Proceedings*.

† Author's closure.

‡ Structural Engr., St. Louis, Mo.

§ Received by the Secretary, February 15, 1929.

load the elements of each moment factor may be summed up into one. This, in most cases, would be warranted either by fact or a very close approximation. The examples should be left as shown, however, because they give very completely and accurately all that is essential for simple cases of continuity within the limits outlined. They are typical of cases with most extensive variations.

For the routine work in a designing office, the computation form shown in Table 13 would possibly serve best. (See Example 1.*)

TABLE 13.—FORM FOR COMPUTATION OF MOMENTS.

	Bending moments.	
Dead load	$67\,\phi \times 22.5^2 = 34\,000\,\phi$	
Live load	$60\,\phi \times 22.5^2 = 30\,500\,\phi$	

MOMENT FACTORS (ϕ).

	Outer span.	Middle span.	Supports.
Dead load	+ 0.084 − 0.003 + 0.007	− 0.089 + 0.015 − 0.039	− 0.091 − 0.005 + 0.013
Total	+ 0.088	− 0.063	− 0.083
Live load	+ 0.084 + 0.007	− 0.089 − 0.039	− 0.091 − 0.005
Maximum	+ 0.091	− 0.078	− 0.096

TOTAL MOMENTS.

Dead load	+ 2 980	− 2 130	− 2 800
Live load	+ 2 760	− 2 360	− 2 930
Total	+ 5 740	− 4 490	− 5 730

A small error occurs, for the support moment only, and this is due to taking the live load at 60 lb. instead of 80 lb. for the center span. The error is less than 1 per cent.

Rapid pencil notations obviously are sufficient, with the multiplications done by slide-rule. The simplest problems thus can be solved in less than 5 min. and the more complicated in less than 10 min.

It has become necessary to emphasize the simplicity and the time element, because so many "simplified methods" have been suggested in the discussion. These are all excellent, and will be referred to in the proper place, but they generally fail to take the final step of producing directly applicable moment factors. While there must be problems which require a great deal of care and

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 719.

a corresponding amount of time, because they are complicated by supplementary restraints and variabilities, the majority of cases should be, and are, elementary enough to be solved by exceedingly rapid methods. Anything less than this would merely induce, as it has in the past, a great amount of rough approximations with corresponding inaccuracies. This is an economic problem; if the cost of designing should run too high, it will be turned into a game of chance.

This is not discounting the many excellent contributions which are of a supplementary nature. They are truly supplementary, because they show how the problems should be solved when they fall beyond the scope of the paper.

Indeed it is difficult to imagine a more graceful, ingenious, and efficient method than that of Hardy Cross,* M. Am. Soc. C. E., or the equally ingenious one by George E. Beggs, M. Am. Soc. C. E., both of which have a general application and each its special, although somewhat overlapping, field of usefulness. Professor Cross' method contains a very slight approximation, but it is entirely too small to consider in view of the exceedingly valuable time-saving procedure.

Professor Cross' method is especially recommended for all cases of continuity, that are not soluble, or should not be solved by means of ready-made moment factors. Professor Beggs' method requires a set of instruments and the manipulation of celluloid models. With the complete equipment at hand its usefulness would be unlimited.

It is particularly gratifying, therefore, that these two methods were introduced by Professors Mylrea† and Parcel,‡ but while this acknowledgment is made with gratitude for the reference and admiration for the methods, it is only fair to state that they offer neither negation nor obsolescence for the writer's contribution.

The paper is elementary in its nature; but none the less it was not intended as a text for junior students. It assumed that the reader would know that there were additional restraints; it also assumed that he would know when the use of the simplified formulas and tables would not be permissible. If the writer took this knowledge for granted erroneously, it has been made very clear by the discussion that there are limits beyond which the formulas and factors should not be used.

As soon as one has to deal with more than three spans, or additional restraints from supporting or supported elements, all possibilities for considerable simplification vanish.

Fortunately, however, the vast majority of continuous beams are of three spans and are freely or almost freely supported. The tables and formulas in the paper are, therefore, an offering to those engineers—and their assistants—who thoroughly know their elasticity problems, but who are none the less looking for a rapid and accurate solution of those that confront them most frequently.

* Available in mimeographed copies for the students of the Univ. of Illinois.
† *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 484.
‡ *Loc. cit.*, January, 1929, Papers and Discussions, p. 162.

In a sense it may be considered an accident that, from the many methods possible, the three-moment theorem was selected as a basis for the paper. It is more than that, however. It is a tribute to a most graceful and accurate solution of an old problem, which has lost nothing by having attained a classical distinction.

From another point of view it stands out with compelling clarity that the one thousand and one (more or less) mathematical masterpieces analyzing stress and strain in elastic frames have had the practical effect of beclouding the elementary issues and setting up—instead of desired simplicity and accuracy—a chimera of faith, hope, and fantasy, which, unfortunately, has been set in a golden frame by committees of authorities and experts. This evil is international, but, curiously enough, it is subject to worship, nationally.

From this viewpoint the method introduced by Professor Cross is particularly significant. From the mass of formulas that are always necessary he has extracted all the essential factors and eliminated others which have insignificant effects on the result and merely tend to confuse the designer in his mathematical operations.

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EVALUATION OF WATER RIGHTS

Discussion*

By John E. Field, M. Am. Soc. C. E.†

John E. Field,‡ M. Am. Soc. C. E. (by letter).§—The writer regrets that the discussors have interpreted the purpose and scope of the paper as being much broader than was intended. There was no thought of discussing water-right values in the humid region, but in the semi-arid West, where the riparian doctrine had been abrogated and the water was subject to appropriation for beneficial purposes, and where because the supply was less than the demand, values were created.

Mr. Jacobs‖ concludes that water rights should be included in appraisals and the writer agrees with him. While in theory the water may belong to the State or the public, the right to its use is a property right. The carrier has a right to the continued carriage and may charge for and derive revenue as a consequence.

If the water right has been acquired by purchase there can be no question of the propriety of including it in an appraisal. In places where it has been filed upon for the use of others (as in a municipal water system belonging to a public service company), it is sometimes contended that, since it belongs to the users—the citizens—such a right should not be included in an appraisal of the company's property; but, in Colorado, the Courts have held that it should be included because the company has the right of carriage. After the point of diversion has been fixed and the quantity and date of priority have been determined, the city cannot change the point of diversion to its own distribution system and deprive the company of its means of revenue. Under the ruling mentioned the value is listed under "carriage right" instead of "water right" and the final result is little changed.

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* Discussion of the paper by John E. Field, M. Am. Soc. C. E., continued from February, 1929, *Proceedings*.

† Author's closure.

‡ Denver, Colo.

§ Received by the Secretary, January 8, 1929.

‖ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2157.

If water that is otherwise running to waste is empounded, it becomes in effect private property. Water that is carried through tunnels or otherwise from one water-shed to another (as is permitted under the decision of the U. S. Supreme Court in the case of Wyoming *vs.* Colorado), is so under control that it, too, is in effect private property. Engineers may safely assume the propriety of including water-right value in general appraisals.

There appears to be one fundamental difference between the writer and most of the discussors in the conception of a water right. Is a water right a tangible or an intangible thing? If it is tangible, then all comment of its inclusion in an appraisal and the necessity of separating it from intangibles, as shown by both Mr. Grunsky* and Mr. Jacobs, disappears. Appraisal of water-right values would be much simplified if instead of using the term, "water right", one used the word, "water" only.

The right to use water is tantamount to its ownership; it should be treated as a physical thing with a value to be established the same as for the physical plant. Often the desire is to obtain the actual benefit derived and the actual value to the owner and such value continues until abandonment, which is not to be contemplated as a possibility in most cases any more than the abandonment of the physical works. For example, in Colorado, many smelters and mills which used water for cooling and other purposes, were abandoned. The value of the physical plant then became nil while the value of the right to the use of the water remained almost undiminished. It could be transferred and used elsewhere for similar, or under certain conditions, quite different purposes. Abandonment through non-use is the only way the right is lost and even then it is a question of intention. The abandoned beet sugar factories furnish another example of the permanence of the value of a water right.

In the case of the smelters, had an appraisal been made before 1893—the year of the demonetization of silver and the beginning of the decline of mining—and had the water been considered as merely a franchise and for that reason a small value was given it, then it would seem that the appraisal of the physical plant should have taken possible abandonment into account, for even then the possibility of the demonetization of silver was under discussion. The abandonment of the water right or its forfeiture or cancellation was, in fact, less probable than abandonment of the physical plant.

The writer's suggestion is that the appraiser get at the truth—the real value—first, and then, as far as the necessities of Court decisions require, alter the nomenclature and form of the appraisal to conform to precedent and let the Court alter any parts that may be in doubt.

In regions where water is so thoroughly in the control of the user as it is in Colorado, where it may be bought, sold, transferred, and its use extended and changed, the conception that water is the product that is owned rather than a water right is thoroughly fixed in the lay mind. While Courts and engineers have discussed "rights" in an academic way, in the final analyses they, too, have treated it as a physical thing. In States where the doctrine

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 491.

of riparian rights has been set aside, and water ownership has been vested in the public, the fact that appropriation to beneficial use "shall never be denied", makes the right a very ponderable thing. The act of appropriation for beneficial use and compliance with the regulations pertaining thereto result in removing the ownership from the public to the individual and only the reversion right remains in the public. Rights-of-way deeds and grants for railroads generally contain a reversion clause and the value of the right of way is allowed in appraisals to the full extent of actual use and necessity, and the value is based on that of similar land in the vicinity. The right of way and the water right in these reversion features are similar.

The propriety of including an indebtedness of $600 000 in the value of the Fort Lyons Canal has been questioned.* The value of the whole property, as determined by taking the market value of a share and multiplying by the number of shares, was $6 000 000, and this was verified by the President of the Company who also informed the appraiser that there was an indebtedness of $600,000 which was being paid off gradually by assessments on the shares. The total value when the debt was paid, in theory at least, would be $600 000 more than it was before.

More details of the method by which the $2 000 000 value of the physical plant was obtained have been requested. There was available the original cost of the work, including interest during construction, engineering, rights of way, etc. Relative costs of labor and material as of date of construction and of appraisal were ascertained. An examination on the ground was made; the cost of the new structures which had replaced the old ones was ascertained; and a value was added by reason of the seasoning of the canal, based on an examination of repairs and upkeep during the first years of operation. Indeed, in this case, the cost of maintenance was at hand for each year since the canal was built. From all these items the cost of replacement was obtained, a small sum was added as value of assemblage, bringing the amount to an even $2 000 000. It was not the Fort Lyon Canal that was being appraised. The appraisal was not a rate-fixing case, nor one to be reviewed by any Court. Its purpose was to ascertain the value of the many water rights of the Company as nearly as possible and approximately correct as a whole rather than as to details; additional expense was not incurred in unnecessary refinements.

Furthermore, the water-right values of three canals (the Fort Lyon and two others) were taken and these were selected for their simplicity of ownership, capitalization, stability of stock values, success in operation, conservatism in management, and progressiveness in improving the character of structures, etc. In short, three of the best managed and most successful canals were selected and the value of a number of water rights of different priorities was obtained and plotted in Fig. 1.† With this curve the probable value of any priority in the same water-shed could be "picked off"; and with this probable value as a base from which to work, the values of the numerous canals, pipe lines, domestic supplies, etc., embraced in the appraisal, were determined.

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2163.
† *Loc. cit.*, April, 1928, Papers and Discussions, p. 1077.

In several cases this probable value was changed as affected by use, soil, locality, abundance or scarcity of water, supply and demand, yield, etc. In fact, the major part of the work of appraisal was not in determining the base curve, but in applying needed corrections. The base curve was valuable, enabling one to show a consistency in results, or good reasons for an apparent inconsistency.

Mr. Jacobs lists* numerous factors that have a bearing on water-right valuation. An example of Factor (*a*) is the difference between the laws of Wyoming and Colorado relative to water being attached to the land. In Wyoming, water may not be transferred and through more economical use, it may not be made to cover a greater area than the permit shows. Except as noted later the title to water goes with the land; its use is comparatively rigid and inflexible. In Colorado, on the other hand, its locus of use may be changed even to diverting it into another canal. It may be made to cover a greater area and, therefore, it is an article which may be sold, the value being largely determined by supply and demand.

Referring to Factor (*b*), in some States there is an irrigation season. In Colorado, the only time limit is the needs of the appropriator, beneficial use being the limitation of the right. There are, also, canals in which water flows during all but the most severe freezing weather. Moreover, water may be temporarily stored to be used later in the season. These extra privileges have value and affect the appraisal.

In States that operate under the Wyoming Code, or modifications thereof, the separating of the water values and land values is not generally necessary; but, in Colorado, the water right must be specifically named in the deed to the land if the intention is to let it go with the land.

The writer approves of Factors (*c*) to (*g*), inclusive, with the supplementary comments by Mr. Jacobs. As to Factors (*h*) and (*i*), it might be added that agricultural uses furnish the alternatives for a domestic supply, and agricultural values influence strongly, if they do not actually fix, the value of a domestic supply. Factor (*j*) appears to be highly improper when the object of the appraisal is rate fixing. It would not be proper, except for purposes of comparison with other methods, when the value could be found by any other method. The process in this case is merely to start with an answer and work back.

Mr. Jacobs and others call attention to the writer's statement† that "whatever amount was paid for * * * old works that were abandoned or destroyed, was added to the cost of the water or water right." The water right was purchased with the knowledge that the old works would not be used, and as far as the purchaser was concerned, the amount he could afford to pay was for the water right only. The seller doubtless included the value of the old works in estimating for what he could afford to sell; but the water right was the only thing of value to the purchaser, so that, in appraising the new works, the value of the water right to the new system of necessity includes the seller's

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, pp. 2158-2159.
† *Loc. cit.*, April, 1928, Papers and Discussions, p. 1071.

estimate of value of the old works which, at the time of appraisal, are non-existent.

It seems that several discussors have confused the value of water stock or shares in a corporation and the value of a water right as used by the writer. The first is the value of shares as shown by numerous sales over a period of years. It is the market value of the shares. The second is the value which is paid for water without physical works and for water to be transferred to some other system. It is the value of a surplus supply owned by some one anxious to sell, and which he can no longer use with reasonable profit; it is a right which, from some cause, he is compelled to sell, possibly from the danger of abandonment and of forfeit; possibly the price is greater than the nature of his own lands and crops justify him in holding.

Mr. Jacobs states:* "As applied to private industry, the logic of capitalizing excess net earnings as a basis for determining water-right values can hardly be questioned." Such a method may be logical, but it might be neither just nor correct. Values as determined by actual sales, by the cost of alternative supplies, and the values in similar enterprises and conditions, might all point clearly to the unreasonableness of an appraisal on such a basis. In cases where all these possibilities of using other methods are absent and there is no power of rate fixing, then such a method of necessity might be used, not as really fixing the value, but as an element in guiding the judgment toward a reasonable valuation. Instead of raising the value of the water right beyond a reasonable figure, a large interest return on the investment as a whole should be shown.

The writer once made an appraisal of water which was used for condensing purposes and was immediately returned to the river. The saving in coal, operating cost, and investment was cut about 33% below the estimated cost of non-condensing engines. Underground water could have been pumped at a relatively small expense; this right of use could have been abandoned and another filing of almost equal efficiency could have been made. The net earnings of the mill, through the use of the water, were enormously increased, making successful competition possible; but the value of the water right in itself was small.

The writer agrees with Mr. Jacobs* that "no single method of valuation can be rigidly applied * * *". In all cases all possible methods should be used; their importance and applicability should be weighted, and the final conclusion determined in accordance with the appraiser's judgment.

In a number of the discussions the writer's statement† that "stored water" has little more value than "unstored water" has been criticized. The writer should have said that "water available for storage" has little more value than that available for direct diversion and use of the same date of priority. Much stored water is sold year by year in Colorado, not always to the same user, but to any one who has need of it. The price is fixed by the supply and demand at the time of sale. In a year of large flow, it has little value. Generally, the

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2161.
† *Loc. cit.*, April, 1928, Papers and Discussions, p. 1072.

seller insists on a reasonable return on his investment in the physical works over a period of years, while the purchaser considers the value to him as an element that helps to grow, save, or increase his crops. When the average charge is great or becomes unreasonable, the use is curtailed by more careful handling and avoidance of waste, or by the construction of additional storage facilities. Such cost at the time of construction is almost the full value of the water, leaving little or nothing as the value of the water right. The gradual advance in values and of returns on such an enterprise accrues to the value of the right.

Mr. Tibbetts* states that: "The few public utilities serving water in California, seldom have a total stock value equal to that of their appraised physical properties * * * although it is recognized that their water rights have substantial values * * *". He does not state whether such utilities have mortgages, bonds, or other debts outstanding. If there is indebtedness then it should be added to the stock value as the writer did in the Fort Lyons case† and, by the method proposed, it might be found that the water right had a value. There are enterprises, the physical costs of which were more than the value of the water delivered, as in some of the reclamation projects. This excess of cost over value must be recognized by the appraiser and corrections made. Such a project should not be taken as a basis for determining values or rights on other projects. On the other hand, if a project, such as this, is being appraised then the method proposed in the paper appears to be eminently suitable. The values of the water right would be determined by examining other projects, and these need not be confined to public utilities, but private, municipally owned, and mutual projects could be used as a basis for constructing a curve of water-right values. The result on a project such as that mentioned by Mr. Tibbetts would show that the investment in physical works was too great for the obtainable revenue and also the amount of the excess.

Mr. Tibbetts states* that, "if he were to purchase a large amount of stock * * * he would first appraise the physical properties and * * * water rights * * *". That is exactly what the writer proposed. He ascertained the values in other projects and applied those values to the properties being appraised.

Mr. Tibbetts‡ "sees no reason why the value of water for domestic purposes should not be based on the city's necessities and on what the city can afford to pay." The reasons are that agricultural water can be purchased for domestic use (as was done in Owens Valley) at agricultural prices, and that there is no practical limit to what a city can afford to pay. Mr. Tibbetts in this connection cites Southern California. If agriculture by irrigation in Southern California should absolutely cease, there would be no need to bring water from great distances for domestic use, and the demand for such use would dwindle with the dwindling population.

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2164.
† *Loc. cit.*, April, 1928, Papers and Discussions, p. 1075.
‡ *Loc. cit.*, September, 1928, Papers and Discussions, p. 2165.

Mr. Tibbetts' remarks under "Land Increment Value",* are particularly pertinent. A bonus to the land owner should be included in "land increment value". At this time in particular it is necessary at least to hold out the hope to the man who takes a raw piece of land and develops it into a going concern, that he will receive a substantial advance in the land value over and above cost and labor. It is the task of determining the size of this bonus that makes land increment value difficult as a basis for determining water values and as a means of allocating the difference in value of adjacent unirrigated land and the irrigated land, to the land, the water, and the bonus. Mr. Hedke† calls this "margin".

In Fig. 2,‡ Mr. Tibbetts shows the average yield and an assumed water demand. It should be recognized that the yield curve determines in large measure the demand curve, because with a given yield curve the user adjusts the character of his crops, methods of farming, type or physical irrigation works, and the acreage irrigated to conform approximately to the yield curve. It is almost invariably true that the yield and demand curves are similar, the difference being more in position than shape, because the demand curve is later in point of time. The construction of reservoirs is the means generally used to make the demand curve and the supply curve more nearly coincident.

Referring to Mr. Scoby's discussion§ it is well recognized that shares, stock, rights, and even measurements of water, such as the inch, vary widely in meaning. To avoid confusion, the writer adopted volume delivered as the measure and he thought he had avoided the necessity of using the variable time element of an irrigation season by adopting the full year as the theoretical maximum. The use of the full year could not have been avoided because many of the rights to be appraised are used for 365 days. Had some arbitrary irrigation season been adopted, such as 180 days, it would have been necessary to give some of the properties a delivery coefficient of 200%, and the meaning of 200 per 100 is paradoxical, confusing, unnecessary, and complicated.

Mr. Hedke states‖ that "it is natural that Mr. Field * * * should complacently open an unsettled subject without ever mentioning ownership." The writer admits also to complacently assuming that values of water rights should be included in appraisals, that they have value, that the user to all intents owns the water, and that they are susceptible of appraisal by fairly rational means, simple in principle, but difficult in application.

Mr. Grunsky states¶ that efforts to find a simple, generally applicable method will be in vain, the main difficulty being in allocating the intangible values to the individual elements. This is true; all appraisals are difficult, but they are based fundamentally on the physical plant which can be appraised in a rational way. To the writer a water right appears to be a physical element, a tangible and not an intangible thing.

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2167.
† *Loc. cit.*, p. 2172.
‡ *Loc. cit.*, p. 2169.
§ *Loc. cit.*, p. 2173.
‖ *Loc. cit.*, p. 2170.
¶ *Loc. cit.*, February, 1929, Papers and Discussions, p. 491.

Mr. Stivers* discusses interestingly various methods of appraisal and the Court decisions thereon. He states that the method proposed in the paper is unsound because it is not a generally accepted method. Any method, however new, untried, or even revolutionary, if it is reasonable, and is evolved in a rational way, will doubtless be accepted by the Courts. It is the duty of the appraisal engineer to use a rational method regardless of other and "generally accepted" methods. The Courts have modified, changed, and even reversed previous opinions too often to cause an engineer to hesitate to use a method he believes to be good. Indeed, the use of a method that is questionable with other methods for comparison and guidance to final conclusions, is advisable. That engineer is wise who studies Court decisions. Through them he obtains ideas and "food for thought"; he finds methods analyzed and learns the reasons for using them; they aid him in developing the judicial attitude; but to say he is bound by such decisions—that he must accept them as final—would be to stop progress in the evolution toward better methods.

Mr. Stivers has given an interesting account of an appraisal of a water right owned by a railroad company in New Mexico, and has used a rational method for its determination. When based, as it is, on an assumed duty of water and on a stated difference in land values before and after the land was deprived of water, the method is correct. The analysis of the result is confined to the correctness of the determination of the duty of water and of the difference in the land values.

It seems that the value of the non-irrigation season flow should have been specifically determined by ascertaining the cost of storage for agricultural purposes, and whether or not such an expense was justified. Deductions in the quantity stored, by reason of evaporation, seepage, and other losses, should be made, and the availability of land and its return in crop value should be studied. The amount for which the railroad company could have sold the water to some other users, and what those users could afford to pay, would fix the value of the winter flow more certainly than if based on the value of the irrigation season supply as was done by Mr. Stivers. If this estimated sale price of the stored water exceeded a reasonable return on the investment plus the cost of maintenance and operation then the winter flow would have a value, and the amount of the excess earnings would be the basis of that value.

In referring to the temporary use of the water, Mr. Stivers states that the farm operations of the railroad do not pay. If the same result should obtain under good management by others (farming undertaken by a corporation as a side line, and as a matter of policy and necessity, seldom pays), then the values given to the winter flow are erroneous unless justified by values obtained by some other method, and some demand other than agricultural.

In conclusion, it appears to the writer that while the laws governing water rights vary in the different States, their customs and decisions are more nearly in harmony. In Wyoming, where the Code differs radically from that in Colorado, the Court decisions are generally in harmony with, if not actually

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 167.

based on, earlier Colorado decisions. Many water rights in Wyoming were acquired under territorial laws. These, in particular, have been treated as, and have a standing similar to, rights in Colorado. The tendency in Nebraska, Kansas, Wyoming, and New Mexico is to follow the Colorado decisions. Practice in Colorado is based largely on decisions, which cover almost every phase of every water-right question. They have undergone the test of time; they have been modified when advisable, and therefore, they present to other Courts a well established and practical solution of most of these questions. While examples and cases applicable particularly to Colorado have been used in argument and illustration herein, they are believed to be largely applicable to other States.

All those who have discussed the paper have shown unusual familiarity with water, its uses, and the problems involved in considering not only values, but uses, customs, laws, and practice. One is tempted to follow to some conclusion the many lines of thought suggested in the discussions; with every sentence more ideas are born, but the writer has resisted this temptation in part and has tried merely to clarify his ideas.

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PAPERS AND DISCUSSIONS

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THE DESIGN OF TALL BUILDING FRAMES TO RESIST WIND

Discussion*

BY H. C. WHITTLESEY, ASSOC. M. AM. SOC. C. E.

H. C. WHITTLESEY,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The writer would like to submit another "short-cut" to calculations of wind stresses.

First.—The total wind moment to be resisted by the framing of any floor is equal to the total wind shear above that floor multiplied by the story height or the average of story heights adjacent to a floor-beam connection.

Second.—The distribution of the moment to the various columns and girders is through the beam connections and is proportional to their ultimate strength. Regardless of how strong the columns or girders are made, the moment developed can not exceed the strength of the connection. If more than the ultimate stress is put on a connection it will slip, and further deflection will not increase the moment in the column or girder. If all girders are equal, grant equal moment to all connections of a floor. If the girders vary in size, as is generally the case, weight the moments proportional to the depths of the beams or connections.

Third.—In an irregularly shaped building there is seldom a typical bent on which to base the wind stress calculations. Assume that the floor system is rigid and that all connections on the floor plan which are oriented with the wind, are working in proportion to their ultimate strengths or to their depths.

The moment in the beams is equal to the moment of the connections. The moment in any column is equal to the sum of the moments of the connections framing to it, but the column moment is divided, part above and part below. Assuming equal moments at both ends of a column in a story, the moment in the column above or below is found by averaging with that in the floor above or below.

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* Discussion on the paper by Albert Ward Ross, Jun. Am. Soc. C. E., and Clyde T. Morris, M. Am. Soc. C. E., continued from February, 1929, *Proceedings*.

† Structural Engr., S. O. Sprager, Los Angeles, Calif.

‡ Received by the Secretary, January 25, 1929.

In addition to these wind stresses, the writer uses the sum of the wind stresses plus $\frac{W\,L}{14}$ times the gravity loads as the basic moment for the design of connections and girders. Allowable gravity stresses are increased 33% for combined wind, or 50% for combined earthquake and gravity stresses. Rivets in tension are considered equal in strength to rivets in single shear and are granted the same increase in stresses; that is, 33% for combined wind, or 50% for combined earthquake and gravity stresses.

The writer treats earthquake stresses in a manner similar to wind stresses by substituting for wind shear the product of the weight of the building, down to and including the floor in consideration, multiplied by a seismic factor. The selection of this seismic factor is "the joker". It is determined largely "by guess and by gosh". The writer uses $\frac{1}{20}$, in Los Angeles, Calif., and vicinity.

Calculations for this method of analyzing the stresses may be arranged as shown in Tables 24 and 25. There are two beam factors, K, for each floor, one for each axis, which when multiplied by the depth of any beam on a floor gives the wind or seismic moment to be taken by the connection at each end of the beam. Wherever gusset connections or haunched beams are used the depth of the connection should replace the depth of the beam in the calculations.

Table 24 is used for finding the beam factors, K. The story height, h, is the average of the story above and the story below any floor level. In determining the seismic force, F, only the dead load or actual load of the building need be considered. The force, F, acting at any floor is the product of the dead weight of the building down to and including the floor in question, by the seismic factor. From this point the calculations are the same for wind or seismic stresses. The total moment per floor, M_T, to be divided among the connections, is F or P times h. The beam factor, K_{NS} (north and south axis), is then found by dividing this total moment, M_T, by N_{NS}.

TABLE 24.—Form for Tabulation of Computations to Determine Value of K.

Floor.	Story height, h.	Dead Load of Building.				Wind shear, P, or $\frac{W_T}{20} = F$.	Total moment per floor, $M_T = Fh$.	Beam Factor.	
		Floors.	Walls.	Total per floor.	Σ, dead load, W_T.			$K_{NS} = \frac{M_T}{N_{NS}}$.	$K_{EW} = \frac{M_T}{N_{EW}}$.

Table 25 is used for finding the divisor, N. Multiply the depth of every beam on a floor, including the spandrels, by the number of times the connection occurs. Consider only those connections which frame to columns.

Enter this product in the proper column, depending on the direction in which the beam is placed. N is then the summation of each column of products. Obviously, there will be two values of N for every floor considered, and values of N are the same for similar floors.

TABLE 25.—FORM FOR TABULATION OF COMPUTATIONS TO DETERMINE VALUE OF N.

Beam No.	Typical Floor.		Second Floor.		Mezzanine Floor.		First Floor.	
	NS	*EW*	*NS*	*EW*	*NS*	*EW*	*NS*	*EW*
$N =$								

In designing the beams of typical floors, due to the increase of stresses allowed for combined stresses, the upper floor sections will be governed by the gravity loads, and only the lower story sections will be influenced by the wind or seismic moments. To determine the floor at which the added moment must be considered, find the maximum moment that the typical floor-beam will resist at the increased stresses, and subtract the end moment of the beam due to gravity loads. If the beam were fully restrained, or fully continuous, the maximum end moment would be $\frac{W\,L}{12}$; but due to probable variations the writer uses $\frac{W\,L}{14}$ as an average value. The loading may be reduced to dead load or actual load when considering seismic stresses. The same end moment is added to connections at exterior columns, due to the relative stiffness of the columns. The resulting difference of moments is the maximum wind or seismic moment that can be carried by the beam. This divided by the depth of beam gives the beam factor, K_{NS} or K_{EW}. By comparing with the K factors in Table 24, the lowest floor at which the beam can be used, is easily picked. From there down the beams of the typical floor will vary at every floor.

By any method of calculation, the sum of the moments about any panel point is zero. Therefore, the sum of the column moments is equal to the sum of the beam moments at any point. Furthermore, if the point of contraflexure is at mid-story height, the moment at each end of a column in any story is the same. To fulfill both these assumptions the column moments must be averaged with the floor above and the floor below. For example, the moment in the column section above the fifth floor is equal to the sum of the moments of the connections framing to the column at the fifth and sixth floors divided by four. Similarly, the moment in the column below is the sum of the moments of the connections of the fifth and fourth floors divided by four. Further simplification is accomplished by averaging the beam factors, K, before multiplying by the beam depths.

When the exterior connections are designed to take the moment, $\frac{W\,L}{14}$ of the beam it is reasonable to take this moment into the exterior columns as well—one-half above and one-half below; in this case the eccentricity of the loading need not be considered.

The floor at which the wind or seismic moment becomes a determining factor in the column design can be found by considering the margin of increase allowed in the fiber stresses. Assuming that the total stress in the column equals $\frac{P}{A} + \frac{M}{S}$; make $\frac{M}{S} = 15\,000 \times 33\%$ (or 50%), and solve for M.

For exterior columns subtract $\frac{W\,L}{14}$ from this moment. Then, twice the residual moment, divided by the sum of the depths of the beams framing to the columns, gives the average value of the beam factor, K, which can be compared with the K-factors of Table 24 for locating the critical floor.

This method was developed primarily for steel frames, but the principles may be applied to a concrete frame by similar reasoning.

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A PROPOSED FORMULA FOR COLUMNS

Discussion*

By T. F. Hickerson, M. Am. Soc. C. E.†

T. F. Hickerson,‡ M. Am. Soc. C. E. (by letter).§—The writer is gratified to note that no one has taken exception to the generality and rationality of Equation (18)‖, which may be written in a more convenient form, as follows:

$$\frac{P}{A} = \frac{f}{u + \sqrt{u^2 - v}} \qquad (47)$$

in which,

$$u = \frac{1}{2} + \frac{1}{2}(\Delta + e)\frac{c}{r^2} + Q;$$

$$v = Q\left[2 - 0.05625\frac{\Delta c}{r^2} - 0.492424\frac{e c}{r^2}\right]; \text{ and,}$$

$$Q = \frac{1}{19.74\,E} f \left(\frac{l}{r}\right)^2.$$

In response to Mr. Prichard's very able discussion,¶ the writer has little to say other than to express indebtedness. With reference to his point that 9.84 instead of 9.87 (or π^2) might be used in the denominator of Equation (8),** the writer would retain the latter value, since it gives an exact theoretical check of Euler's formula when the slenderness ratio becomes very large. However, if the term, $\frac{e}{8}$, in Equation (8) is changed to $\frac{e}{7.92}$, there is a still closer check with rigid theory; so that, when $\frac{l}{r} = 100$, etc.,†† y_t and y_0

* Discussion of the paper by T. F. Hickerson, M. Am. Soc. C. E., continued from December, 1928, *Proceedings*.

† Author's closure.

‡ Prof. of Civ. Eng., Univ. of North Carolina, Chapel Hill, N. C.

§ Received by the Secretary, February 21, 1929.

‖ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1440.

¶ *Loc. cit.*, December, 1928, Papers and Discussions, p. 2721.

** *Loc. cit.*, May, 1928, Papers and Discussions, p. 1438.

†† *Loc. cit.*, p. 1439.

are identically the same $\left(0.3690\ \frac{c}{r^2}\right)$. This slight revision in the coefficient of the e-term is incorporated in the general formula, Equation (47), thus making it perfectly rational, so far as all practical applications are concerned.

The remarks made by Dr. Salmon* are highly appreciated, but the writer finds reasons for rejecting his proposed straight-line formula.† It is entirely too approximate except for very small values of the slenderness ratio, as will be seen by comparison with the more exact Equation (47).

Let $\frac{\Delta\, c}{r^2} = 0.0015\,\frac{l}{r}$; $\frac{e\, c}{r^2} = 0.1$; $E = 30\,000\,000$; and $f = 36\,000$. If $\frac{l}{r} = 80$, Equation (47) gives $\frac{P}{A} = 24\,165$; while Dr. Salmon's formula* gives $\frac{P}{A} = 28\,000$, which is 16% greater than the correct value. If $\frac{l}{r} = 100$, the results are 19 845 as compared with 27 170, which is 37% greater; and if $\frac{l}{r} = 120$, the results are 15 750 and 26 390, respectively, or 67% greater by using Dr. Salmon's formula.

As to the point made concerning fixed-ended columns,‡ the writer concurs. In dealing with columns having considerable restraint at the ends (but not fixed rigidly), a "free length" of $\frac{2}{3}\ l$ was assumed for use in the formula for pivoted ends. Obviously, any other "free length" ranging between $\frac{l}{2}$ and l may be used in the formula (or table) for pivoted ends to represent special conditions where the ends are known to be more or less restrained.

The writer cannot accept the statement† made by Professor Ross that the solution of the secant formula by trial and error is quicker than the solution of Equation (18). In order to compare the time required to solve Equations (27)§ and (47), the following data were used: Given $\frac{l}{r} = 100$; $\Delta = 0$; $e = \frac{1}{10}\,r$; $\frac{c}{r} = 1.5$; $E = 30\,000\,000$; and $f = 20\,000$; it is required to find the value of $\frac{P}{A}$.

Making the proper substitutions in Equation (27), the writer proceeded thus: (1) Assuming $\frac{P}{A} = 12\,000$, the formula gave $\frac{P}{A} = 15\,650$; and (2) assuming $\frac{P}{A} = 15\,000$, the formula gave $\frac{P}{A} = 14\,894$, which is sufficiently close as a check.

The writer considered himself lucky in guessing the second value; and a calculating machine, together with a table of secants, was available, but the time consumed in making the two trial solutions was 25 min. Then, the direct solution of Equation (47), giving $\frac{P}{A} = 14\,924$, was done in 6 min.

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* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2726.
† "Columns," Lond., 1921, p. 43, and Fig. 5.
‡ *Loc. cit.*, p. 2727.
§ *Loc. cit.*, May, 1928, Papers and Discussions, p. 1446.

Professor Ross admits that it is strictly correct to deal separately with the initial deflection due to the bent shape of the column, but on account of a supposed simplicity, he states a preference for the secant formula which assumes all the eccentricity at the ends of the column. If a certain initial bow is tolerated, it does not seem necessary that it be changed 24% in order to eliminate one term in a general expression, especially since no complications are involved. The value, e, is logically a function of r, whereas Δ is a function of both l and $\frac{l}{r}$; and they should be treated separately, for this reason if for none other. The interesting paper quoted by Professor Ross,* entitled "Equivalent Eccentricities Due to Secondary Stress", should be read in full to obtain a clear understanding of the theory involved.

The Hunley formulas presented by Mr. De Blois† give high allowable stresses, and they evidently are supposed to apply only to existing railway bridges. Equation (46),‡ for $\frac{e}{r} = 0.1$, $\frac{c}{r} = 1.5$, and maximum stress = 20 000, gives a close agreement with rigid theory for $\frac{l}{r} = 50$ and $\frac{l}{r} = 100$; but for $\frac{l}{r} = 150$, the allowable stress is 26% greater than the correct value; and for $\frac{l}{r} = 200$, it is 68% greater. Therefore, the claim made by Mr. De Blois§ "that the [Hunley] formula follows the secant formula remarkably close up to a slenderness ratio of $k = 200$", cannot be justified.

As to the importance of the ratio, $\frac{c}{r}$, which Mr. De Blois discusses, it is evidently an integral part of a general column formula; but the writer is inclined to think that a fixed average value of $\frac{c}{r}$ (1.5, for example) should be used in a practical working formula, although it may not sometimes appear to be on the side of safety. Greater values of $\frac{c}{r}$ imply that a relatively smaller amount of the material is further removed from the axis of bending; and the maximum edge stresses are increased somewhat (see Fig. 10), but the tips of flanges can safely stand a greater stress intensity than the entire face of a flange, as tests have demonstrated.‖ The curves in Fig. 10 are based on the following values: $e = 0.1\,r$; $\Delta = 0.00001 \left(\frac{l}{r}\right) l$; $E = 29\,000\,000$ lb. per sq. in.; yield point = 33 000 lb. per sq. in.; and the apparent factor of safety = 2.

The eccentricity of loading is more logically a function of r than of c, h, or l, as used by Mr. Hunley.

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* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2729.
† *Loc. cit.*, p. 2730.
‡ *Loc. cit.*, p. 2733.
§ *Loc. cit.*, p. 2732.
‖ Progress Rept. of the Special Committee on Steel Column Research, *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 357.

The writer is pleased to have Mr. Godfrey's endorsement of Equation (18).* Its agreement with the straight-line formula, $16\,000 - 70\,\frac{l}{r}$, for values of the slenderness ratio under 120 is not so close, however, as Fig. 9† shows. In assuming no eccentricity of loading and an initial bow of $\frac{1}{300}\,l$ (a linear deflection), the resulting curve is unduly straight.

The initial bow depends on the slenderness ratio as well as on the length. Thus, in comparing two columns with the same length, the one having the larger slenderness ratio will bend more; just as, in the case of two columns with the same $\frac{l}{r}$, the one with the larger l is expected to bend more.

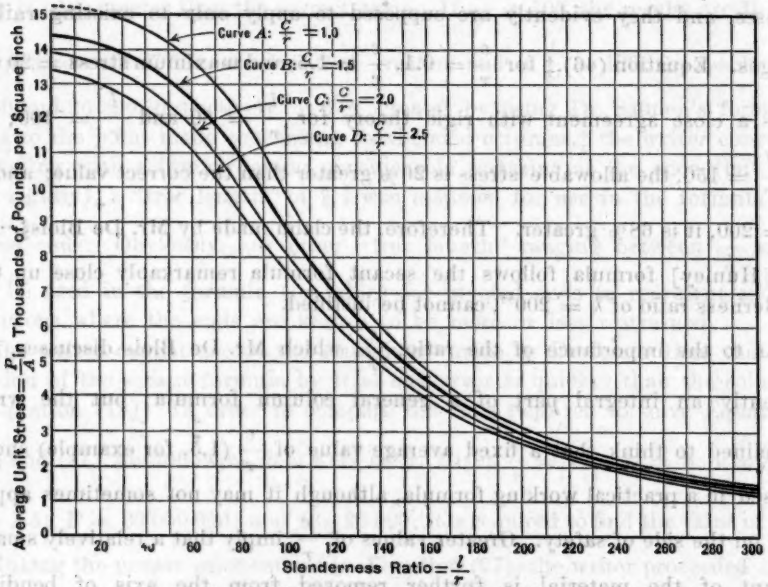

FIG. 10.—AVERAGE THEORETICAL ALLOWABLE COLUMN STRESSES FOR VARIOUS VALUES OF THE RATIO, $\frac{c}{r}$.

The writer is convinced that the one mathematically correct formula for structurally permissible columns is a reversed parabolic curve, and not a straight line, as stated by Mr. Godfrey.† On the basis of pivoted ends, the curves in Fig. 10 were drawn from the solution of Equation (47). Curve *B* coincides almost exactly with Curve *A*, in Fig. 3‡; and the simplified approximate formula,

$$\frac{P}{A} = 14\,500 - \left(\frac{l}{r}\right)^2 \left[0.8 - 0.0025\,\frac{l}{r}\right] \ldots\ldots\ldots\ldots (48)$$

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2733.
† *Loc. cit.*, p. 2735.
‡ *Loc. cit.*, May, 1928, Papers and Discussions, p. 1443.

can be used accurately and safely for all values of the slenderness ratio up to 200. Thus, if $\frac{l}{r} = 100$, $\frac{P}{A} = 9\,000$ lb. per sq. in.

The relative central deflections of the pivoted column can be determined from Equation (10),* which may be written:

$$f = \frac{P}{A}\left[1 + \frac{y_t}{l}\left(\frac{l}{r}\right)\left(\frac{c}{r}\right)\right] \quad \ldots\ldots(49)$$

or,

$$\frac{y_t}{l} = \frac{\frac{f}{\frac{P}{A}} - 1}{\left(\frac{l}{r}\right)\left(\frac{c}{r}\right)} \quad \ldots\ldots(50)$$

The solution of Equation (50) for seven different values of the slenderness ratio is given in the last column of Table 3. For this purpose, $f = 33\,000$ lb. per sq. in.; $e = \frac{1}{10}r$; $\Delta = \frac{1}{100\,000}\frac{l}{r}l$; $\frac{c}{r} = 1.5$; and $E = 29\,000\,000$. In studying the table, it is interesting to note that the terms, $\frac{e}{l}$ and $\frac{\Delta}{l}$, are due to initial conditions, while the term, $\frac{y_1}{l}$, is the additional relative deflection due to column action.

TABLE 3.—Relative Central Deflections of a Pivoted Column.

$$\left(\text{When } f = 33\,000 \text{ lb. per sq. in., and } e = \frac{1}{10}r, \quad \Delta = \frac{1}{100\,000}\left(\frac{l}{r}\right)l; \frac{c}{r} = 1.5; \text{ and } E = 29\,000\,000\right).$$

$\frac{l}{r}$.	$\frac{e}{l}$.	$\frac{\Delta}{l}$.	$\frac{y_1}{l}$.	Total, $\frac{y_t}{l}$.
20	0.00500	0.0002	0.00026	0.00546
40	0.00250	0.0004	0.00063	0.00353
60	0.00167	0.0006	0.00123	0.00350
80	0.00125	0.0008	0.00217	0.00422
100	0.00100	0.0010	0.00371	0.00571
120	0.00083	0.0012	0.00547	0.00750
200	0.00050	0.0020	0.01280	0.01530

For the case of restrained ends, the allowable stress may be determined satisfactorily by using Curve *B*, Fig. 10, with the proper "free length" value of l. If, for example, a "free length", l', of $\frac{3}{4}$ l is assumed as simulating columns in trusses, then for $\frac{l}{r} = 120$, the slenderness ratio to be used in Fig. 10 is $\frac{l'}{r} = \frac{3}{4} \times 120 = 90$, and $\frac{P}{A} = 9\,900$.

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1439.

For restrained columns in trusses and buildings, in which the probable "free length" is $\frac{3}{4}\ l$, the following formula automatically gives results agreeing closely with Curve *B*, Fig. 10, for all values of the slenderness ratio up to 200:

$$\frac{P}{A} = 14\,500 - \left(\frac{l}{r}\right)^2 \left[0.45 - 0.001\ \frac{l}{r}\right] \ldots\ldots\ldots\ldots (51)$$

In solving Equation (51), the total length of l (not l') should be used. Thus, if $\frac{l}{r} = 100$, $\frac{P}{A} = 11\,000$ lb. per sq. in. To check this value against Curve *B*, Fig. 10, $\frac{l'}{r} = \frac{3}{4} \times 100 = 75$ and $\frac{P}{A} = 11\,370$.

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ENGINEERING FOUNDATION COMMITTEE ON ARCH DAM INVESTIGATION

ARCH DAM INVESTIGATION

Discussion*

By Messrs. B. A. Smith, E. Probst, and Lars R. Jorgensen.

B. A. Smith,† M. Am. Soc. C. E. (by letter).‡—Engineering Foundation and the Committee on Arch Dam Investigation have placed the engineering world under a debt of gratitude which future generations must discharge by making full use of the information so admirably collected and so exhaustively discussed. In this discussion the writer has followed the Committee's notation, as far as possible, except that F§ is used instead of V for shear.

A striking feature of the investigation is that disclosed in Fig. 86,‖ namely, the deformation of the crest into a wave form with two side crests and a central trough. This corresponds to an inextensional deformation of the cylinder which, although involving relatively large displacements, has only a very small effect on the stresses. This mode of deformation is described in Example 1, and the writer shows therein that the shrinkage which must occur during the setting of the concrete is sufficient to account for the down-stream deflection of the crest at the center and the up-stream deflection near the abutments.

Example 2 gives the displacement at all points of the crown generator and the corresponding stresses when the reservoir is full, the temperature being constant and equal to the initial temperature. These values are summarized in Table 45.

* Discussion on the Report of the Committee of Engineering Foundation on Arch Dam Investigation, continued from January, 1929, *Proceedings*.

† Civ. and Hydr. Engr., Melbourne, Victoria, Australia.

‡ Received by the Secretary, November 15, 1928.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2038.

‖ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 133.

Column (8) of Table 45 gives the displacement at all points of the crown generator treated as forming part of a complete cylinder, while Column (9) gives the displacement (or deflection) due to the inextensional deformation of the cylinder. Column (10) gives the resultant of these two displacements and Column (11) the observed deflection. The discrepancy between the calculated and observed displacement at Elevation 30 and thereabouts is no doubt due to some inaccuracy in the assumed boundary conditions at that elevation.

TABLE 45.—RESERVOIR FULL; TEMPERATURE CONSTANT.

Elevation, in feet. (1)	ζ, in feet. (2)	x. (3)	t, in feet. (4)	$\frac{E_0}{w}\frac{u_1}{R^2} = \lambda_1$. (5)	$\frac{M}{w} = \mu_1$. (6)	$\frac{F}{w} = \nu_1$. (7)	u_1, in inches. (8)	u_2, in inches. (9)	$u_1 + u_2$, in inches. (10)	U', in inches. (11)	T, in pounds per foot of height. (12)	M, in foot-pounds per foot. (13)	F, in pounds per foot. (14)
60.0	0.0	−2.75	2.00	− 0.81	0	0	−0.02	+0.37	+0.35	+0.37	− 10 000	+ 0	0
55.0	5.0	−0.38	2.00	+ 1.97	18	− 7	+0.04	+0.33	0.37	0.38	+ 25 000	+ 1 100	− 400
50.0	10.0	0.00	2.00	+ 4.81	62	− 10	+0.10	+0.30	0.40	0.39	+ 60 000	+ 3 900	− 600
45.0	15.0	1.38	2.00	+ 7.80	115	− 10	+0.17	0.27	0.44	0.39	+ 97 000	+ 7 200	− 600
40.0	20.0	2.75	2.00	+11.08	151	− 3	+0.24	0.24	0.48	0.39	+138 000	+ 9 400	− 200
35.0	25.0	4.13	2.00	+14.72	132	+ 13	+0.32	0.21	0.53	0.38	+184 000	+ 8 200	+ 800
30.0	30.0	5.50	2.00	+18.67	0	+ 42	+0.40	0.18	0.58	0.36	+233 000	0	+ 2 600
28.2	31.8	6.00	2.18	+17.18	− 87	+ 54	+0.37	0.17	0.54	0.33	+239 000	− 5 400	+ 3 400
24.6	35.4	7.00	2.54	+14.12	−307	+ 66	+0.30	0.14	0.44	0.30	+227 000	−19 200	+ 4 100
21.0	39.0	8.00	2.90	+10.87	−531	+ 54	+0.24	0.12	0.36	0.24	+197 000	−33 200	+ 3 400
17.4	42.6	9.00	3.26	+ 7.41	−656	+ 9	+0.16	0.10	0.26	0.20	+151 000	−41 000	+ 600
13.8	46.2	10.00	3.62	+ 3.75	−540	− 81	+0.08	0.08	0.16	0.15	+ 85 000	−33 700	− 5 000
10.2	49.8	11.00	3.98	0.00	0	−225	0.00	0.06	0.06	0.09	0	0	−14 000

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 131, Fig. 84.

Example 3 gives the displacements and stresses due to a fall of temperature of 1° cent., the reservoir being empty. The agreement between the calculated and the observed displacement is reasonably close. In this case no inextensional deformation is superposed because there is no applied force tending to press the dam against its abutments.

Attention should be drawn to the regions of tension at the crest and at the base (see Tables 45 and 46). From Table 45 it will be seen that at the crest there is a tension of 10 000 lb. per ft. of depth, and from Table 46, if there is a fall of temperature of 10° cent., there is a further tension of 5 600 lb. per ft. This is a total tension of 15 600 lb. per ft., or, since the dam is 2 ft. thick, $\frac{15\,600}{2 \times 144} = 54$ lb. per sq. in., which may account for a crack that developed.

Again, from Tables 45 and 46 it will be seen that if there is a temperature drop of 10° cent., the tension at Elevation 10 is 160 000 lb. and at Elevation 14 there is 105 700 less 85 000, or 20 700 lb.; that is, at the base there is 290 lb. per sq. in., and at Elevation 14, 40 lb. per sq. in., which is sufficient to account for the observed vertical crack at the base.

The writer's remarks have been confined to the discussion of the experimental data, but he would like to express his keen appreciation of the valuable experimental work of Professor Beggs,* and the important analytical work of

* *Proceedings*, Am. Soc. C. E., May, 1928, Part 3, p. 219.

Professor Westergaard* on the general lines of which future progress must be made.

For rough practical purposes, the writer has made frequent use of the solution for the complete cylinder given in his paper on "Arched Dams".† The results are satisfactory both for the wedge-shaped section and for cylindrical tanks of uniform thickness.

TABLE 46.—THE EFFECT OF A FALL IN TEMPERATURE OF 1° CENTIGRADE, WITH RESERVOIR EMPTY.

Elevation, in feet. (1)	ζ, in feet. (2)	x. (3)	t, in feet. (4)	$\lambda_2 = \frac{u}{U_0}$. (5)	$\mu_2 = M\frac{R^2}{E_0 U_0}$. (6)	$\nu_2 = F\frac{R^2}{E_0 U_0}$. (7)	$u = \lambda_2 U_0$. (8)	U_0, in inches. (9)	$u - U_0 = u'$, in inches. (10)	U' observed, in inches. (11)	T, in pounds per foot of height per degree centigrade. (12)	$M = \mu_2 \times \frac{E_0 U_0}{R^2}$ U_0, in feet. (13)	$F = \nu_2 \times \frac{E_0 U_0}{R^2}$. (14)
60	0.0	−2.75	2.00	+0.070	0	0	−0.0010	−0.0140	+0.013	+0.013	− 560	0	0
55	5.0	0.00	2.00	+0.045	− 1.54	+ 0.5	−0.0006	−0.0140	0.013	0.013	− 360	+ 60	− 20
50	10.0	0.00	2.00	+0.016	− 5.31	+ 0.8	−0.0002	−0.0140	0.014	0.014	− 128	+ 212	− 32
45	15.0	0.00	2.00	−0.026	− 9.83	+ 0.7	+0.0004	−0.0140	0.014	0.015	+ 208	+ 392	− 28
40	20.0	0.00	2.00	−0.093	− 12.95	+ 0.2	+0.0013	−0.0140	0.015	0.015	+ 744	+ 518	− 8
35	25.0	0.00	2.00	−0.191	− 11.31	− 1.2	+0.0027	−0.0140	0.017	0.015	+ 1 528	+ 452	+ 48
30	30.0	5.50	2.00	−0.315	0	− 3.6	+0.0044	−0.0140	0.018	0.014	+ 2 520	0	+144
28.2	31.8	6.00	2.18	−0.213	+ 7.40	− 4.6	+0.0030	−0.0140	0.017	0.014	+ 1 880	− 299	+184
24.6	35.4	7.00	2.54	−0.002	+ 26.00	− 5.5	0.0000	−0.0140	0.014	0.013	+ 20	−1 046	+220
21.0	39.0	8.00	5.90	+0.224	+ 44.70	− 4.4	−0.0031	−0.0140	0.011	0.010	− 2 700	−1 788	+176
17.4	42.6	9.00	3.26	+0.469	+ 54.60	− 0.6	−0.0066	−0.0140	0.007	0.007	− 6 110	−2 184	+ 24
13.8	46.2	10.00	3.62	+0.730	+ 44.50	+ 6.8	−0.0102	−0.0140	0.004	0.004	−10 570	−1 760	−272
10.2	49.8	11.00	3.98	+1.000	0	+18.6	−0.0140	−0.0140	0.000	0.008	−16 000	0	−744

Example 1.—The Deformation of a Section of a Cylinder without Stretching.—Suppose $O\ V$ (Fig. 166), is the vertical axis of a thin cylinder, $A\ B\ A'\ C$, of radius, $V\ A = V\ B = V\ A' = O\ C = r$. Consider the par of the surface, $A\ C\ A'\ B$, bounded by two inclined planes, $O\ C\ A$ and $O\ C\ A'$, each making an angle, β, with a horizontal plane through $O\ C$, and suppose that $O\ V\ B\ C$ is a vertical reference plane and that the angle, $A\ V\ B = A'\ V\ B = \alpha$. The position of any point, P, on the surface is defined by its height, y, above O (or C), and its azimuth angle, ψ, from the reference plane, $O\ V\ A\ C$. If P is on the boundary, $A\ C$, then,

$$y = r \tan \beta \sin \psi \qquad (127)$$

This relation may be regarded as defining the boundary, $A\ C$. After deformation, P goes to an adjacent point P', so that the component distances from P are (see Fig. 167): $P\ N = u$ (above P); $N\ M = v$ (to the right of P); and $M\ P' = w$ (toward the axis). This w corresponds to the u in the writer's paper on "Arched Dams".†

Suppose Q is a point near P on the vertical generator through P, and that R is a point near P on the horizontal arc through P, so that $Q\ P\ R$ is a small

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 231.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

right-angled triangle. In Fig. 167, this triangle is shown greatly enlarged for the sake of clearness. After deformation, the triangle, $P\,Q\,R$, goes to $P'\,Q'\,R'$.

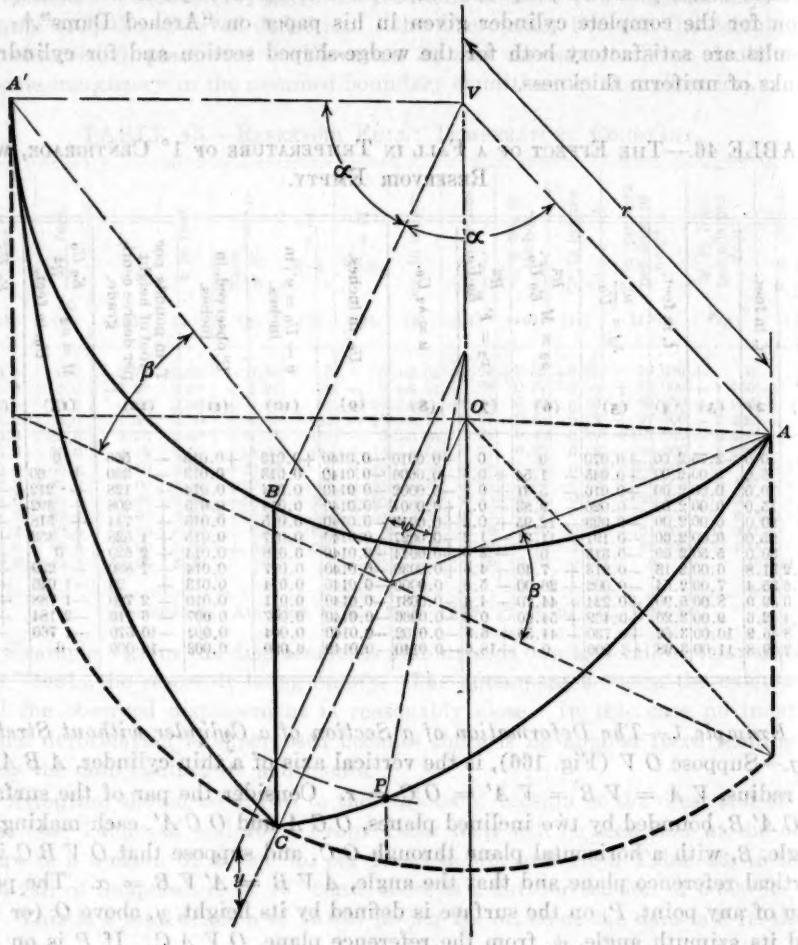

FIG. 166.

The conditions that there shall be no stretching stipulate that: $P'\,Q' = P\,Q$; $P'\,R' = P\,R$; and the angle, $Q'\,P'\,R' = Q\,P\,R$.

Expressed in symbols, these conditions are:

$$\left.\begin{aligned} \frac{d\,u}{d\,y} &= 0 \\ \frac{d\,v}{d\,\psi} - w &= 0 \\ \frac{d\,v}{d\,y} + \frac{l}{r} \times \frac{d\,u}{d\,\psi} &= 0 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (128)$$

A solution of these equations is required such that when $\psi = 0, u = v = \frac{dw}{d\psi} = 0$, and along the foundation (that is, when $y = r \tan \beta \sin \alpha$), $w = 0$. The condition, $\frac{dw}{d\psi} = 0$, avoids a discontinuity about the generator, $\psi = 0$.

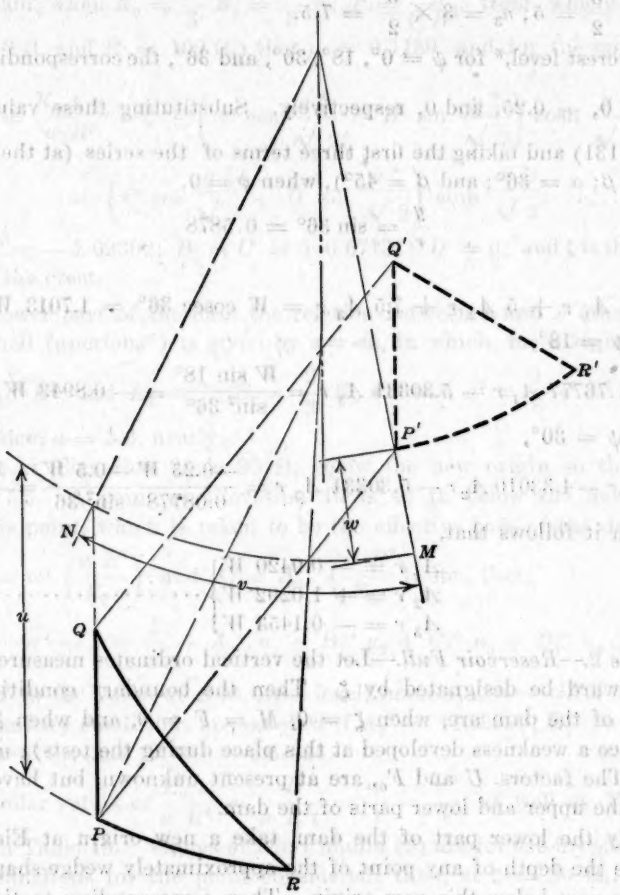

FIG. 167.

If W is the displacement of the crest at $\psi = 0$, the required solution is:

$$u = -\frac{W(\psi - \sin\psi)}{\tan\beta \sin^2\alpha} - \sum \frac{r}{n} A_n (1 - \cos n\psi) \quad \ldots\ldots\ldots (129)$$

$$v = \frac{W}{\tan\beta \sin^2\alpha}\left[(1 - \cos\psi)\frac{y}{r} - \frac{1}{2}\tan\beta\left(\psi - \frac{1}{2}\sin 2\psi\right)\right] + \sum r A_n \left[\left\{\frac{y}{r} - \frac{n^2}{n^2-1}\tan\beta\sin\psi\right\}\sin n\psi + \frac{n}{n^2-1}\tan\beta(1 - \cos\psi\cos n\psi)\right] \quad \ldots (130)$$

$$w = \left(\frac{y}{r} - \tan\beta\sin\psi\right)\left[\frac{W\sin\psi}{\tan\beta\sin^2\alpha} + \sum n r A_n \cos n\psi\right] \quad \ldots (131)$$

In Equations (129), (130), and (131), the values of n are chosen so that $n\,\alpha = p\,\frac{\pi}{2}$, in which, $p = 1, 2, 3 \ldots\ldots$

In the Stevenson Creek Dam, $\alpha = \frac{\pi}{5}$, so that, $n_1 = 1 \times \frac{5}{2} = 2.5$; $n_2 = 2 \times \frac{5}{2} = 5$; $n_3 = 3 \times \frac{5}{2} = 7.5$.

At the crest level,* for $\psi = 0°$, 18°, 30°, and 36°, the corresponding values of $\frac{w}{W}$ are 1, 0, — 0.25, and 0, respectively. Substituting these values of w in Equation (131) and taking the first three terms of the series (at the crest, $y = r \sin\alpha \tan\beta$; $\alpha = 36°$; and $\beta = 45°$), when $\psi = 0$,

$$\frac{y}{r} = \sin 36° = 0.5878$$

and,

$$2.5\,A_1\,r + 5\,A_2\,r + 7.5\,A_3\,r = W \operatorname{cosec} 36° = 1.7013\,W$$

When $\psi = 18°$,

$$1.76777\,A_1\,r - 5.30331\,A_3\,r = \frac{W \sin 18°}{\sin^2 36°} = -0.8943\,W$$

and when $\psi = 30°$,

$$0.64705\,A_1\,r - 4.33010\,A_2\,r - 5.30331\,A_3\,r = \frac{-0.25\,W}{0.08778} \frac{-0.5\,W}{\sin^2 36} = -4.2952\,W$$

from which it follows that,

$$\left.\begin{aligned} A_1\,r &= -0.9420\,W \\ A_2\,r &= +1.0292\,W \\ A_3\,r &= -0.1453\,W \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (132)$$

Example 2.—Reservoir Full.—Let the vertical ordinates measured from the crest downward be designated by ζ. Then the boundary conditions in the upper part of the dam are, when $\zeta = 0$, $M = F = 0$, and when $\zeta = 30$ ft., $M = 0$ (since a weakness developed at this place during the tests); $u = U$; and $F = F_0$. The factors, U and F_0, are at present unknown, but have the same values for the upper and lower parts of the dam.

To study the lower part of the dam, take a new origin at Elevation 50, and let z be the depth of any point of the approximately wedge-shaped section forming the part below this new origin. Then corresponding to the junction of the upper part of a uniform thickness of 2 ft., and the lower part of a thickness of t (given by $t = c\,z$, in which, $c = 0.1$ and z is greater than 20), $\zeta = 30$ ft. when $z = 20$ ft. The axis of z, and, therefore, also of x, is vertically downward, and $z = a\,x$, in which, $a = 3.6$ very nearly.

In the lower part of the dam the boundary conditions are: when $z = 20$ ft. (that is, $x = 5.5$), $u = U$, $M = 0$, and $F = F_0$; and when $z = 40$ ft. (that is, $x = 11$), $u = 0$, and $M = 0$. In all, there are ten equations for ten unknowns, namely, the four arbitrary constants for the upper part, the four for the lower part, and the two unknowns, U and F_0.

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, Fig. 86, p. 133.

Using these conditions and writing $u = \frac{u'\ w\ R^2}{E_0\ t}$; $A = \frac{A'\ w\ R^2}{E_0\ t}$, etc., in which, w is the weight of 1 cu. ft. of water, that is, 62.5 lb., the following expressions are derived: For a complete cylinder, $\mu^4 = 12\ \frac{E_0}{E_1}\ \frac{1}{t^2\ R^2}$, and, for a segmental dam, when $E_0 = \frac{2}{3}\ E_1 = \frac{2}{3}\ E$, $\mu^4 = \frac{8}{t^2\ R^2}$, from which it follows (since $t = 2$ ft. and $R = 100$ ft.) that $\mu = 0.1189$, and for the upper part of the dam,

$$w = \frac{E_0\ t\ u}{w\ R^2} = \zeta + \left(A' \cos \frac{\mu\ \zeta}{\sqrt{2}} + B' \sin \frac{\mu\ \zeta}{\sqrt{2}}\right) \cosh \frac{\mu\ \zeta}{\sqrt{2}} + \left(C' \cos \frac{\mu\ \zeta}{\sqrt{2}} + D' \sin \frac{\mu\ \zeta}{\sqrt{2}}\right) \sinh \frac{\mu\ \zeta}{\sqrt{2}} \quad \ldots\ldots\ldots\ldots (133)$$

in which, $A' = -1.62302$; $B' = C' = +0.67137$; $D' = 0$; and ζ is the depth, in feet, below the crest.

In the lower part of the dam the relation between z and x (the argument of the Michell functions*) is given by $z = ax$, in which, for a segmental dam, $a^2 = \frac{1}{12}\ \frac{E_1}{E_0}\ c^2\ R^2$, and $E_1 = E$, $E_0 = \frac{2}{3}\ E$; so that, $a^2 = \frac{1}{8}\ c^2\ R^2 = \frac{1}{8}\ (10)^2 = 12.5$; hence, $a = 3.6$, nearly.

A point at Elevation 30 is 20 ft. below the new origin so that for this point, $x = 5.5$. A point at Elevation 10 is 40 ft. below the new origin so that for this point, which is taken to be the effective base of the dam, $x = 11$. Writing $u = u''\ \left(\frac{w\ R^2}{E_0\ c}\right)$, and $A_1 = A_1''\ \left(\frac{w\ R^2}{E_0\ c}\right)$, etc., then,

$$w'' = 1 - \frac{l'}{x} + A_1''\ u_1 + B_1''\ u_2 + C_1''\ u_3 + D_1''\ u_4 \ldots\ldots (134)$$

in which, l' is the value of x at crest level, namely, $l' = -2.75$.

The boundary conditions are satisfied if $A_1'' = 14.30950$; $B_1'' = +5.74175$; $C_1'' = -0.88746$; and $D_1'' = -8.52325$.

The tabular values of $\frac{E_0\ u}{w\ R^2} = \frac{T}{w\ R\ t} = \frac{u'}{t} = \frac{u''}{c}$, as well as T, M, and F, are given in Table 45. Values of x in Column (3) are for use in connection with Michell's functions for the point considered; thus, at $\zeta = 30$ ft., $z = 20$ ft. $= 3.6\ x$, so that $x = 5.5$, nearly.

Equation (133) gives the values of $\frac{E_0\ u}{w\ R^2}$; for this part, $t = 2$ ft., $E_0 = \frac{2}{3}\ E = \frac{2}{3} \times 3\ 600\ 000$ lb. per sq. in., or 345 000 000 lb. per sq. ft.

Equation (134) gives the value of $\frac{E_0\ u}{w\ R^2}$ for the lower part below Elevation 30. In this part, $c = \frac{1}{10}$, and since R is given in feet and $w = 62.5$ lb. per cu. ft., u will be expressed in feet. It is reduced to inches in Tables 45 and 46, however.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), Table 1, p. 2052.

The values of T in Table 45, Column (12), are computed from the equation*:

$$T = \frac{E_0\, t}{R} \cdot u \qquad (135)$$

The values of M for the upper and lower parts of the dam are derived by double differentiation from Equations (133) and (134), respectively. For the upper part, ζ is between 0 and 30 and,

$$M = \frac{w}{\mu^2}\left[-A_1' \sin\frac{\mu\zeta}{\sqrt{2}}\sinh\frac{\mu\zeta}{\sqrt{2}} - B_1'\left(\sin\frac{\mu\zeta}{\sqrt{2}}\cosh\frac{\mu\zeta}{\sqrt{2}} - \cos\frac{\mu\zeta}{\sqrt{2}}\sinh\frac{\mu\zeta}{\sqrt{2}}\right)\right] \qquad (136)$$

For the lower part, x is between 5.5 and 11 and,

$$M = w\,a^3\,x^3\left[\frac{5.5}{x^3} + A_1''\frac{d^2u_1}{dx^2} + B_1''\frac{d^2u_2}{dx^2} + C_1''\frac{d^2u_3}{dx^2} + D_1''\frac{d^2u_4}{dx^2}\right] \qquad (137)$$

The values of M (positive when the generators are bent concave to the axis), are given in Table 45, Column (13).

The values of the shear, F, across each foot width of the dam (measured along a horizontal tangent) in the neighborhood of the central section is given in the upper part by,

$$F = \frac{w}{\mu\sqrt{2}}\left[A_1'\left(\sin\frac{\mu\zeta}{\sqrt{2}}\cosh\frac{\mu\zeta}{\sqrt{2}} + \cos\frac{\mu\zeta}{\sqrt{2}}\sinh\frac{\mu\zeta}{\sqrt{2}}\right) + 2\,B_1'\sin\frac{\mu\zeta}{\sqrt{2}}\sinh\frac{\mu\zeta}{\sqrt{2}}\right] \qquad (138)$$

and, in the lower part, by,

$$F = -w\,a^2\left[A_1''\left(x^3\frac{d^3u_1}{dx^3} + 3x^2\frac{d^2u_1}{dx^2}\right) + \ldots\right] \qquad (139)$$

The values of F are given in Table 45, Column (14). A positive value of F indicates that M is decreasing as ζ increases.

In Equations (136) and (138), the constants have the same value as in Equation (133), and those in Equations (137) and (139), correspond with Equation (134).

Example 3.—Temperature Stresses with Reservoir Empty.—The boundary conditions are: At the crest, $M = F = 0$; at $\zeta = 30$ ft., $u = U_1$, $F = F_1$, and $M = 0$ (in which, U_1 and F_1, are at present unknown), for both the upper and lower parts; and at the base, $x = 11$, $u = U_0$, and $M = 0$.

As in Example 2, there are ten equations for ten unknown quantities. The equation for the displacement, u, is from $\zeta = 0$ to 30 ft.:

$$\frac{u}{U_0} = A_2'\cos\frac{\mu\zeta}{\sqrt{2}}\cosh\frac{\mu\zeta}{\sqrt{2}} + B_2'\left(\sin\frac{\mu\zeta}{\sqrt{2}}\cosh\frac{\mu\zeta}{\sqrt{2}} + \cos\frac{\mu\zeta}{\sqrt{2}}\sinh\frac{\mu\zeta}{\sqrt{2}}\right) \qquad (140)$$

in which, $U_0 = \frac{3}{2}\,e\,R\,\theta = \frac{3}{2} \times 0.0000077 \times 100 \times 12 \times 1° = -0.014$ in. for a 1° cent. drop in temperature.

* A negative sign indicates tension.

The negative sign indicates that the designer must (1) forcibly displace all points of the base through a distance of 0.014 in. outward from the axis; and, then, (2) contract the whole surface through the same distance, which will not alter the stresses and will bring the base back to its original position: $A_2' = +0.06957$; $B_2' = -0.02878$; and, for the lower part, from $x = 5.5$ to 11:

$$u = A_2'' u_1 + B_2'' u_2 + C_2'' u_3 + D_2'' u_4 \ldots\ldots\ldots\ldots (141)$$

in which, $A_2'' = -17.08784$; $B_2' = -8.34623$; $C_2' = +4.26342$; and $D_2' = +8.49452$. Column (8) of Table (46) gives the value of u in terms of U_0 corresponding to the first operation; Column (9) shows the uniform contraction applied to all points in the second operation. Column (10) gives the resultant of these two operations, while Column (11) gives the observed values. The agreement is fairly good, except near $\zeta = 30$ ft.

The thrust, T, corresponding to the displacement, u, given by Equations (140) and (141) is given by the equation:

$$T = \frac{E_0\, t}{R} u \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (142)$$

and is tabulated in Column (12) of Table 46.

The bending moment, M, from $\zeta = 0$ to 30, is given by,

$$M = \frac{E_0\, U_0\, t}{\mu^2\, R^2} \left[-A' \sin \frac{\mu\,\zeta}{\sqrt{2}} \sinh \frac{\mu\,\zeta}{\sqrt{2}} - B' \left(\sin \frac{\mu\,\zeta}{\sqrt{2}} \cosh \frac{\mu\,\zeta}{\sqrt{2}} - \cos \frac{\mu\,\zeta}{\sqrt{2}} \sinh \frac{\mu\,\zeta}{\sqrt{2}} \right) \right] \ldots\ldots\ldots\ldots\ldots\ldots (143)$$

and, from $x = 5.5$ to 11, by,

$$M = \frac{E_0\, U_0\, c}{R^2} a^3 x^3 \left(A_2'' \frac{d^2 u_1}{d x^2} + B_2'' \frac{d^2 u_2}{d x^2} + C_2'' \frac{d^2 u_3}{d x^2} + D_2'' \frac{d^2 u_4}{d x^2} \right) \ldots (144)$$

They are tabulated in Column (13), Table 46, the constants being as previously given. The shear between $\zeta = 0$ and 30, is given by,

$$F = \frac{E_0\, U_0\, t}{\mu\, R^2 \sqrt{2}} \left[A_2' \left(\sin \frac{\mu\,\zeta}{\sqrt{2}} \cosh \frac{\mu\,\zeta}{\sqrt{2}} + \cos \frac{\mu\,\zeta}{\sqrt{2}} \sinh \frac{\mu\,\zeta}{\sqrt{2}} \right) + 2\, B_2' \sin \frac{\mu\,\zeta}{\sqrt{2}} \sinh \frac{\mu\,\zeta}{\sqrt{2}} \right] \ldots\ldots\ldots\ldots\ldots\ldots (145)$$

and, between $x = 5.5$ and 11, by,

$$F = -\frac{E_0\, U_0\, c}{R^2} a^2 \left[A_2'' \left(x^3 \frac{d^3 u}{d x^3} + 3 x^2 \frac{d^2 u}{d x^2} \right) + \ldots\ldots \right] \ldots\ldots (146)$$

They are tabulated in Column (14), Table 46.

The tilting of the dam from its original vertical position at the base* is possibly due to deformation without stretching, as discussed in Example 1. This deformation has no effect on the stresses because (1) the generators remain straight so that the bending moment, M, is zero throughout; (2) since M is zero throughout, the shear, F, also vanishes; and (3) since the stretch along a horizontal arc vanishes, the arch thrust, T, is also zero.

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 193.

The agreement between the calculated deflections given in Column (10), Table 45, after allowing for the added deflections due to the tilting of the dam and the observed deflections shown in Fig. 126,* and Column (11) is fair except at Elevation 30, where there is some uncertainty as to the exact conditions. It may be shown that the deflections at the center line (shown in Fig. 127†), for a head of water of 50 ft., are more nearly in accordance with the values given in Table 45, Column (10), and are greater than those shown in Fig. 126 for a head of 60 ft.

The writer suggests that more weight should be given to the direct measurements of the deflections than to the values of the moments and shears calculated from the deflections. In particular, he would point out that the tilting of the dam at the base indicated by the deflection diagrams in Figs. 126, and 127 points to a zero bending moment rather than to the considerable bending moment suggested in the diagrams.

The existence of a considerable tension at the crest (as shown in Tables 45 and 46, Column (12)), and at the base (as shown in Table 46), probably accounts for the vertical cracks observed at these places on the center line.

DR. ING. E. PROBST‡ (by letter).§—The experiments at Stevenson Creek have made clear to the writer, certain necessary modifications for other designs under similar conditions. As a result of temperature variations, especially just after removing the forms and filling the reservoir to within 60% of the total height, the bond between the foundation rock and the concrete structure was broken, for all practical purposes, on the up-stream side,‖ so that there was no possibility of developing tension along that plane. About two days after pouring the upper one-sixth of the arch there was absolutely no bond left between rock and concrete.

This shows the need for vertical joints, extending from face to face in radial lines, as already used in many arch dams. These joints should be keyed and then sealed with a bent copper sheet on the water side. After the concrete has hardened all the joints must be completely filled. For this reason it is best to make joints relatively large so that they may be more readily filled with a sand-cement mortar. Neat cement mortar is not recommended because of its high rate of shrinkage. Arch action is possible only if the joints are completely filled.

The cracks that developed along the up-stream side of the dam show that the assumption of restraint at the abutments must be accepted with caution. If, in checking the computations for an arch dam, a tensile stress is indicated at the supports—and in a small dam these are avoidable—the project should be re-designed so as to ensure only compression. According to Fig. 136,¶ there seems to have been only compression at the reactions in the upper half of the dam. At a point half way down there was a compressive stress of about 1 000

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 174.
† *Loc. cit.*, p. 175.
‡ Prof., Technische Hochschule, Karlsruhe, Baden, Germany.
§ Received by the Secretary, January 2, 1929.
‖ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 113.
¶ *Loc. cit.*, p. 188.

lb. per sq. in. at the supports, while from Fig. 94* it may be noted that the stress on the water side must have been zero if not in tension. To change this into compression the thickness of the arch should be made greater. According to the theory of restrained arches, such an increase would inevitably cause tensile stresses at the supports, but since it is not possible to transmit such stress to the foundation rock, there is an increased increment of compression on the down-stream face. In a certain sense this places the designer in a dilemma. The situation would become still more unfavorable if an allowable compressive stress of less than 560 lb. per sq. in. is adopted and the arch would need to be thickened accordingly.

The writer recommends that the arch dam be supported somewhat as shown in Fig. 168 (*c*). The haunches on the down-stream side can be designed so that no crack will occur on the opposite face. In addition, the haunches reduce the unit bearing pressure at the sides. They are particularly necessary when it is desired to place reinforcing steel at the ends.

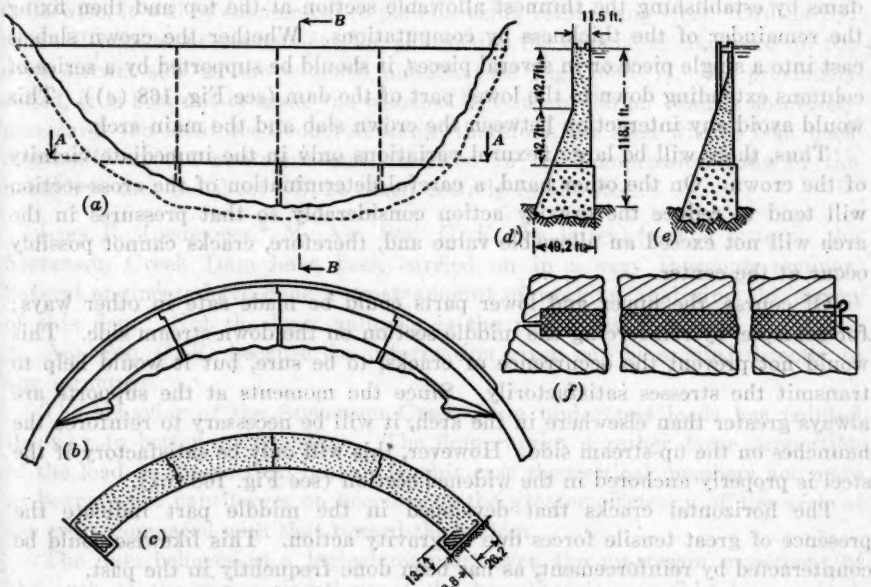

FIG. 168.

Furthermore, for arches with a central angle of less than 50°, every effort should be made to avoid arch action and unfavorable static conditions at points of restraint. This conclusion is founded on the observation (Fig. 95*), that a crack occurred in the lower third of the dam near the center of the down-stream face. This crack doubtless had an unfavorable influence on the horizontal beam and arch action.

With reference to the combined effect of arch action and horizontal beam action on the one hand and gravity effect on the other, the Stevenson Creek

* *Proceedings*, Am. Soc. C. E., May, 1923, Pt. 3, p. 138.

Dam essentially verified results obtained by theoretical analysis. The differences in the upper and lower third of the dam are explained by the presence of the cracks.

As might be expected, the influence of gravity was greatest in the lower third. Because of this influence, the center of the dam rose as much as 0.05 in. from the foundation rock on the up-stream side. Nevertheless, the arch action with the horizontal beam action was great enough to develop the crack on the down-stream face. Thus, it is apparent that in the lower third, the dam was not effective in producing perfect gravity action or arch action. One of the two effects should have been removed so that the dam could have been built to provide complete and satisfactory resistance by the remaining factor.

Another crack developed about in the middle of the upper third on the down-stream face and extended completely through the dam at the crown. This may be attributed, in addition to other actions, to the fact that the dam was of constant thickness for the upper 30 ft. Such cracks may be avoided in arch dams by establishing the thinnest allowable section at the top and then fixing the remainder of the thickness by computations. Whether the crown slab is cast into a single piece or in several pieces, it should be supported by a series of columns extending down to the lower part of the dam (see Fig. 168 (*e*)). This would avoid any interaction between the crown slab and the main arch.

Thus, there will be large flexural variations only in the immediate vicinity of the crown. On the other hand, a careful determination of the cross-section will tend to reduce the gravity action considerably so that pressures in the arch will not exceed an allowable value and, therefore, cracks cannot possibly occur at the center.

Of course, the upper and lower parts could be made safe in other ways; for example, by reinforcing the middle section on the down-stream side. This would not prevent the occurrence of cracks, to be sure, but it would help to transmit the stresses satisfactorily. Since the moments at the supports are always greater than elsewhere in the arch, it will be necessary to reinforce the haunches on the up-stream side. However, this will only be satisfactory if the steel is properly anchored in the widened section (see Fig. 168 (*c*)).

The horizontal cracks that developed in the middle part indicate the presence of great tensile forces due to gravity action. This likewise could be counteracted by reinforcement, as has been done frequently in the past.

If it becomes desirable to insure that the dam be subjected to all gravity or all arch action a method suggested would be as follows: Consider a dam in a canyon such as Fig. 168 (*a*), with about 120 ft. of water behind it. The valley is a trapezoidal form which is well suited for an arch dam. Let the width of the valley be about 360 ft. at the crest elevation.

The dam proposed consists essentially of three parts: A lower wall made of a number of blocks separated by joints; a very plastic intermediate strip (Fig. 168 (*f*)); and an upper wall in the form of a full-centered arch with variable thickness. In the lower part, arch action may be avoided most readily by making joints wide enough. It is advisable to key the joints and seal them on the water side with a bent strip of copper.

The horizontal strip (indicated to an enlarged scale in Fig. 168 (*f*)), should be of a material, such as asphalt felt, that will permit small relative horizontal and vertical displacements between the upper and lower parts of the dam. For the upper part of the dam the section shown in Fig. 168 (*d*) is better than that in Fig. 168 (*e*). For a profile, such as Fig. 168 (*a*), the lower section should be about one-third the length of the dam. If the valley is quite deep and V-shaped, the two sections should be of more nearly equal height. In some cases it may be found economical to make the lower part higher than the upper.

In designing the upper part or arch section of the dam it is first necessary to approximate the value of its central angle. For the greatest convenience in construction this should be as large as the valley profile will permit with a minimum of 100 degrees. The permissible variation from this minimum depends entirely on the conditions in the valley and the relative economy.

A plain concrete arch will be found most economical when the canyon is 400 ft., or less, wide at the crest elevation and the height of the dam is 130 ft. Otherwise, it will be economical to plan on using reinforcing steel. Ordinarily, this will only be necessary near the abutments, the rods being anchored on the down-stream side in the widened haunch (see Fig. 168 (*c*)). As previously stated, it is best not to make the widened section any larger than is absolutely necessary. In reinforced concrete dams, therefore, the steel will be bent back diagonally at the abutment and anchored in the compression zone of the haunch so that the shear will be taken entirely by the steel.

LARS R. JORGENSEN,* M. AM. SOC. C. E. (by letter).†—The tests on the Stevenson Creek Dam have been carried on in a very thorough manner. Several engineers have done a great amount of work in analyzing the action of this dam. Not the least valuable is the paper‡ by Raymond E. Davis, M. Am. Soc. C. E., because the information given can be applied directly to any structure.

The behavior of the Stevenson Creek Dam under test loads has pointed the way to better arch designs. The dam carries a rather large proportion of the load on the vertical beam. In this case the vertical members act more as beams than cantilevers on account of the greater efficiency of the arch at the crest, compared with that toward the bottom.

The dam behaves in a logical manner; even the up-stream movement of the arch at the upper elevations at the haunches is as might be expected. The action is like that of a long column with failure impending. Judging from the amount and shape of the deformation, it is almost certain that complete failure will take place considerably before the crushing strength of the material is reached.

The failure of the Stevenson Creek Dam is presumably not imminent, but enough cracks and other indications of weakness have appeared to show that the dam is somewhat in distress, although the maximum unit stress is approximately only 1 200 lb. per sq. in. This condition should not happen on

* Cons. Engr., Constant Angle Arch Dam Co., San Francisco, Calif.

† Received by the Secretary, February 13, 1929.

‡ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 199.

a commercial dam and need not happen. The dam was made with the intention of destroying it and, therefore, it was proportioned in a manner different from that of a commercial arch dam. At, and toward, the crest it subtends a large enough angle to make arch action most effective, but the arch is too long in relation to its thickness; hence, the **S** shape of the horizontal deflection curve of the arch in its upper parts. At low elevations the subtended angle becomes very small; the arch becomes too flat for best arch action, and an unusually large amount of load is thrown on the vertical beam between the bottom and sides of the canyon and the top arch rings where arch action is effective. This is more nearly vertical beam action than ordinary cantilever action (see Figs. 48 to 53*). Since these vertical beams are the longest in the middle they load the top part of the arch more at this point, and, undoubtedly, have considerable influence in shaping the deflection curve like an **S** toward the crest.

The strain curves show the material most heavily stressed (horizontally) to be at the abutments in the zone between Elevations 30 and 40 where arch action is at its best. This indicates plainly that a correctly designed arch dam should be thicker at the abutments than at the crown in order to be stressed more equally in all its parts.

The writer is acquainted with several tests on models made by B. E. Torpen, M. Am. Soc. C. E., that seem to prove that actual failure in an arch of equal horizontal thickness will not take place until the crushing strength of the material has been exceeded, provided the subtended angle is 60° or more and the length of the arch is not more than thirty to forty times its thickness. In such an arch—removed somewhat from the foundation—the deformation curve would not be **S**-shaped and the plastic deformation at high stresses can be counted on to minimize the bending stresses.

The crest length of the Stevenson Creek Dam is 140 ft., and the thickness, 2 ft. The ratio of slenderness is, therefore, 70 and the arch cannot be expected to hold together until the crushing strength of the material has been reached, which is also indicated by the shape of its horizontal deflection curve. This is not a true arch dam; it is too flat near the bottom and is only efficient as an arch between Elevation 30 and the crest.

Professor Westergaard's analysis† of the structural action of the Stevenson Creek Dam is very interesting in that it shows the presence of large twisting couples. On model tests by Mr. Torpen, the failures looked more as if pieces had been twisted from the remainder of the body than as if the material had been crushed.

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* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, pp. 98-102.

† *Loc. cit.*, p. 231.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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COUNTY SEWER DISTRICT WORK IN OHIO AND ASSESSMENT OF COST ACCORDING TO BENEFITS

Discussion*

BY MESSRS. FRANK C. TOLLES AND E. G. BRADBURY.†

FRANK C. TOLLES,‡ M. AM. SOC. C. E. (by letter).§—This paper is devoted, first, to an outline of the provisions of the statutes which enable sanitary improvements in suburban Ohio; and, second, to a description of a procedure of assessment. The Enabling Act mentioned by the author is purely local, but in so far as it may be used as a guide for others, it is proper to comment on certain phases of it.

The failure of the 1913 statutes to include provisions for detailed and specific advice to property owners of the terms and of the costs of the undertaking, led to objection on constitutional grounds and resulted in later amendment. The law now requires as a precedent to critical legislation, that there shall be prepared detailed plans, estimates of cost, and tentative assessments. Individual notice of these is given to each property owner whose address is known. In a project of wide scope this procedure is burdensome. However, and despite the burden of the preparatory work, improvements have been carried on under the Revised Act, although possibly with stricter regard for proved needs. Irrespective of its legal aspect, the statute requirement affords sound protection to the property owner, and it has served to restrict the more speculative developments which, for a time, threatened to bring Ohio sanitary districts into disrepute. From the administrative side, it would be desirable if the preparatory engineering could be lessened. Particularly is this the case in the smaller counties which do not maintain sanitary engineering departments, and in which interest in suburban development is sporadic.

It is noteworthy that the provision of the law for assessing interest charges instead of making these a levy against income, serves to lessen operation

* Discussion of the paper by E. G. Bradbury, M. Am. Soc. C. E., continued from February, 1929, *Proceedings*.

† Author's closure.

‡ Civ. Engr. with George B. Gascoigne, Cleveland, Ohio.

§ Received by the Secretary, January 26, 1929.

costs, and thus favors the direct consumer or user. This feature is helpful in establishing the district as a going concern.

There is still in the law a provision for paying to the county commissioners who authorize the improvement, certain fees which are proportioned on the improvement cost. These fees are small (one-third to one-tenth of 1%, with a fixed maximum), but it would be better if the payments were not so intimately related to the size of the project.

To Mr. Bradbury's statement* of the extent of improvements which have been installed under the Ohio law, should be added the fact that, in one or two counties, the auditors' records carry some rather long lists of delinquents in the payments of assessments under the District Act. This fact may reflect a temporary condition in a local real estate market, or it may indicate a too rapid and unwise extension of improvements. This comment is not directed to Franklin County, in which Mr. Bradbury is County Sanitary Engineer.

The Method of Assessing.—The major consideration in the paper concerns a method of apportioning construction costs.† Although developed to fit particular statutes and precedents, the method is mathematically, and probably legally, applicable in the general case of spreading costs over a district. To those who have studied this subject, Mr. Bradbury's contribution is welcome as helping to clarify a very "muddled" and unsatisfactory situation. It is, however, open to certain criticisms of practice.

Assessments which proceed from the powers of the Government must be levied in the manner prescribed by enabling laws. The statutes which cover the County Sewer Districts of Ohio recite that, in the case of main works, property which abuts the improvement may be assessed for local service, and that the remaining cost, less any county share, shall be spread as a district assessment on all property in the district and in accordance with special benefits. Other Ohio statutes which relate to municipal improvements provide that special assessments be levied (1) by percentage of tax value; (2) in proportion to benefits; or (3) by the foot front. There is a further provision in the Municipal Code that the sum total of special assessments within a 5-year period shall not exceed one-third the value of the property after the improvements are made. Assessments for other purposes or by other Governmental units are generally by benefits. The mass of legal decisions is found in municipal operations.

In Ohio, the tendency has generally been to apportion assessed costs in proportion to some readily established physical characteristic, including area, frontage, distance from the improvement, or a combination of these. In cases where levies have been made on the basis of benefits, the computation has sometimes been roughly estimated in terms of area or other characteristic and then modified to meet the judgment of the assessor. In other cases, the so-called "point system" has been used to yield a proportionate and relative estimate of benefits. For small improvements on which the assessors are well acquainted with individual properties this latter method affords reasonably good results.

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2061.
† *Loc. cit.*, p. 2067.

The Bradbury Formula.—Whereas assessments spread to physical characteristics are readily determinable and comparatively simple of application, those based on benefit must be distributed by exercise of judgment and to meet several factors, which in the aggregate, at least, are rather intangible. Mr. Bradbury's method does not eliminate judgment, but through itemization of elements, focuses judgment on a particular point. In essence, the plan is to make the assessment proportionate to the product of: (1) Ultimate cost less future expenditure; (2) a factor of deferred use; and (3) the square root of a value ratio.

As an expression of individual opinion there is no reason why this method should not be as good as any other reasonable rule. When it is coupled with assessment for local service on a frontage or area basis, this method admits influence by different variables. It, therefore, minimizes the inequities that result from distributing the assessed cost according to any single characteristic. However, any conclusion that assessments computed by Mr. Bradbury's method are more equitable than those determined by physical characteristics alone, is predicated on the assumption that the fundamental data are reasonably accurate. In spreading assessments as described by the author, it is necessary to evaluate certain factors, thus:

(*a*) The total ultimate cost of the improvement when—at some future and unknown date—it is completed.

(*b*) The time sequence—perhaps by 3 or 5-year increments—when different parts of the improvement will be required, and this for a total future period of, say, 20 years.

(*c*) The present value of the land which is to be assessed.

The first of these factors—the eventual cost—is within the range of reasonable estimation, although there should be taken into account the occasional practice, in rural development, of preceding sanitary improvements with pavements. It is conceivable, however, that such estimates would average within an error of 15 per cent.

The time element is rather more difficult to fix; that is to say, for example, that a given sub-district will require trunk outlets or plant facilities in 1945. An error in assigning a sub-area to a 10-year time zone instead of to a 15-year zone might lead to a unit assessment which departs by about 20% from that which might prove more equitable. A rough visualization of this is afforded by comparing the partial assessments in Column (9) of Table 2,* for the approximately similar zones *E* 1, *E* 2, and *E* 3, which represent 2-year time differentials.

Assessments are admittedly based on judgment, and it is perhaps not fair to subject the expression of such judgment to more or less mathematical scrutiny, for Mr. Bradbury is undertaking not to point off the decimal, but rather to make determinations rational and comparable. However, the writer has recently made some assessment studies in which the estimate of the time factor was attempted, and he was struck with the discordances of opinion among those whose judgment was pertinent. Faced with the question, "When will this particular area need sewerage", well-informed officials, engineers, and conservative realtors gave answers that differed by 10 years, and that further

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2070.

yielded no very clear indication of a preponderance of opinion. In small areas, and with a well marked trend of development, better accord would be expected.

In the matter of setting a value (Mr. Bradbury's third factor), there exists considerable opportunity for variation. Since valuation is itself an expression of judgment, rather than an agreed or barter price, it is subject to all the influences of personal error whether it be from tax duplicate or the result of special appraisal. Also, is the fact that county sewer district improvements are sought by areas which are in process of change from quasi-farm land to sections of suburban development. This betokens a state of flux with a corresponding effect on values, and the resulting difficulties in valuation are great. Moreover, such valuations are those of the moment, during a period when changes are rapid and when there is a wide spread between tax values and holding prices. Neither is it true that, irrespective of the movement of values, their ratios remain unchanged. County sewer districts are generally composed of relatively valuable land which is in immediate and clear need of sanitary improvement, and, further, of acreage which is still under cultivation. The latter is often a large part of the gross area. It is included in the district wisely and (under the present law) in bulk willingly, but the fact remains that its need for service is distinctly of the future. Because of this, there is excellent reason for including valuation or a similar factor as an element of assessment, thus increasing the share to high value lands. However, it is questionable whether, in such cases, values can be estimated to greater precision than with direct evaluation of the resulting benefits.

Table 3* shows derived acreage benefits in a maximum ratio of about 15 to 1 (Areas F5 and A4). This is a greater difference than would result from an apportionment by area, but the writer has spread assessments which have a unit ratio of 12 to 1 without introducing the factor of value.

Conclusion.—Mr. Bradbury has based his plan on the thesis that benefits are measured by the increase in value which results from the improvement. This is true, of course, but it is not at all clear that the enhancement is proportionate to the initial value or to any constant factor thereof.

The writer is impressed with the paper as a workable plan that accounts for pertinent elements in a manner which reflects the matured judgment of an engineer who has labored with the problem. However, the theory is not—and cannot be—provable. Under conditions in which the variables are determinable within reasonable error, this method should yield satisfactory results, but its use should certainly be coupled with experience and caution in fixing futurities as well as present valuations.

The general subject of special assessments offers a splendid field for engineering analysis and economic thought.† However, a technical passion for precision should not obscure the human elements that are involved.

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* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2071.

† An excellent view of current practice in assessing is afforded by bibliographies which are contained in Engineering Societies Library Searches 383, 383a, 2897, and 3555. The Library of Congress also has similar lists. The legal side is covered by the works of Dillon ("Law of Municipal Corporations"), McQuillan (similar title), and, for Ohio, by Page and Jones ("Taxation by Assessment"). Most of the current references bear on assessments for paving.

E. G. BRADBURY,* M. AM. SOC. C. E. (by letter).†—The rates charged by the City of Columbus for disposal of sewage from county sewer districts quoted by Mr. Bonney,‡ being based on the number of connections, have the merit of minimizing the necessary accounting between city and county and the work incident to the assessment of the charges against individual property owners. Such rates are not, of course, absolutely equitable but perhaps, on the average, they represent the value of service rendered as correctly as do more complicated methods.

The writer has observed no evidence that unincorporated territory provided with sewer and water improvements is likely to resist annexation to the city. More than one-half the area in which these facilities have been installed by the county has already been annexed to Columbus, and there has been little, if any, opposition by residents or property owners.

The curve for estimating the relation of maximum sewage flow to the average, mentioned by Mr. MacDowell,§ is also used by the City Engineering Department of Columbus. This fact explains the apparent inconsistency in the statement of Mr. Bonney that the capacities provided by the city are substantially equal to those developed by the basis proposed by the writer, although there is a wide discrepancy between the amounts allowed for infiltration by the two methods. The Morse curve represents the relation of maximum flows to normal quantities in the particular cases observed. The maximum flow is much greater than can be accounted for by fluctuations in the domestic flow and always occurs during or after storms. Therefore, it is evidently due to excessive infiltration or surface water, and is not a function of the domestic flow, as it is made by the use of the curve. It follows that the estimated maximum is unduly affected by the forecast of future population and water consumption. For example, if an area of 100 acres is assumed to have a probable future population of 20 per acre, with a water consumption of 75 gal. per day per capita, and an allowance of 750 gal. per acre per day is made for leakage, the maximum flow as shown by the curve being four times the average, the total quantity per acre is found to be,

$$20 \times 75 = 1\,500 \text{ (domestic)}$$
$$1\,500 + 750 \text{ (leakage)} = 2\,250$$
$$2\,250 \times 4 = 9\,000 \text{ gal. per day}$$

After deducting 225% of the domestic flow as a reasonable maximum rate, the actual allowance for infiltration is found to be 5 625 gal. per acre per day.

By the same process, if the population of the district is taken at 30 per acre and the water consumption at 100 gal. per capita per day, and the same figure, 750 gal. per acre per day, used for leakage, it will be found that provision is made for 8 250 gal. per acre per day of infiltration.

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* County San. Engr., Columbus, Ohio.

† Received by the Secretary, February 18, 1929.

‡ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 251.

§ *Loc. cit.*, p. 254.

If the total normal flow is approximately equal to that on which the curve is based, it will give satisfactory results, but otherwise its use may lead to serious errors. Much of the published information concerning infiltration is misleading because it refers to average leakage rather than to the much higher rate following severe storms. Infiltration is often referred to as being quite constant in quantity, which is far from true. It appears more rational to estimate separately the maximum domestic and infiltration rates, adding for commercial and industrial use when necessary, than to set up an arbitrary relation between average and maximum without consideration of the components.

Mr. MacDowell questions* the use of valuation as a factor in apportioning assessments. The writer agrees that this factor cannot be used "with surety that justice is being served," but contends that this is true of any method used, and that justice will be more nearly and more often served if value is taken into consideration than otherwise. The measure of benefit is the enhancement of value of the property. Property located in unfavorable surroundings, and, therefore, low in price, cannot be enhanced in value (in terms of dollars, not in terms of percentage) as much as well-located lots which command prices perhaps several times as great. The use of a factor in direct proportion to value over-emphasizes this condition, but some recognition of relative value is necessary. It is true that a wholly illogical standard of values has been established for platted areas as compared with unplatted land, and owners of the latter may change its status almost over night, thus gaining some advantage over their less astute neighbors. On the other hand, it is to be noted that the owners of platted and developed territory are usually the instigators of the improvement and, in a sense, force it on those whose property is still in acreage and who, for reasons of their own, have not yet seen fit to put it on the market. Among such owners will be found many who are looking forward to later development and others who desire to occupy their land for agricultural or other purposes. There is some justice in placing the greater burden of assessment on those who are asking for the improvement as compared with those who consider it more or less premature in relation to their holdings and accept it only as a necessary evil.

It is desirable that the assessment on the more remote areas, often held in sizable parcels as farm land, be made as low as possible consistent with reasonable equity, and the use of the value factor helps in this direction. There have been several cases in Ohio Courts in which it has been claimed that sewer assessments were confiscatory, and at least two cases in which the Courts have sustained this contention. The writer believes that such a result will often follow if the valuation factor is eliminated, but can hardly occur if value is properly recognized.

The Missouri Sewer District Law, passed in 1927,† provides for the levying of taxes in sewer districts, and for re-adjusting the apportionment from time to time as conditions change. This is an interesting development and,

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 257.

† Laws, Missouri 1927, p. 439.

unless too cumbersome in operation, may be an improvement over the Ohio method.

Mr. Olson* gives a graphic description of conditions where suburban development is not controlled by proper laws and regulations. Similar conditions (perhaps in a lesser degree) were encountered and cured when the Ohio law came into operation. Real estate operators still sometimes prefer to install sewers and water mains in their developments, but under the Ohio law they are required to do such work according to approved plans, and under county supervision and inspection, the owner paying all expense. In Franklin County such owners are further required to convey such sewers and water mains to the county as soon as completed, for a nominal consideration, for the use and benefit of their property; to place in the County Treasury an amount representing the estimated cost of maintenance for one year; and to authorize the county to levy maintenance assessments against the same property as may be necessary in the future. This procedure was adopted after it was learned by experience that such improvements, if left in the hands of the original owner, are usually entirely neglected after the lots have been sold out, no one being responsible for their maintenance and the county having no funds for the purpose, nor any legal right to operate private sewers or water mains.

Mr. Tolles† refers to a provision in the Ohio law which authorizes the payment to County Commissioners of fees based on the cost of the work. This section has been held unconstitutional by the Franklin County Courts, the decision being affirmed by the Circuit Court of Appeals, and is, therefore, no longer effective in this Judicial District, which includes several counties in Central Ohio. How this decision affects the remainder of the State has not been determined.

The various uncertainties and difficulties in establishing time and value factors, referred to by Mr. Tolles,‡ must, of course, exist, but the writer has not found them serious. Assumptions are necessary in many classes of engineering work. The fact that one must estimate future population, water consumption, and infiltration does not justify an arbitrary guess at the necessary capacity of a trunk sewer. The same principle may reasonably be applied to the assessment problem. Intelligent forecasting of the various factors, and a logically developed result therefrom, should ordinarily produce the most nearly correct answer to problems of this type.

In conclusion, the desirability of a method of apportioning assessments to the satisfaction of property owners is self-evident. The Franklin County method has the merit of appealing to the general public as fair and equitable; it is capable of quick explanation in simple language, and provides something more to rely on than mere personal judgment.

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 509.
† See p. 1022.
‡ See p. 1023.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE PRACTICAL UTILITY OF HIGHWAY TRANSPORT SURVEYS

Discussion*

By N. W. Dougherty, M. Am. Soc. C. E.

N. W. Dougherty,† M. Am. Soc. C. E. (by letter).‡—Highway transport surveys, as described by the author, have a special value in connection with the construction of highway bridges that are to be financed by toll charges. In 1927, the State Legislature of Tennessee authorized the construction of sixteen toll bridges. The legislation comprises three acts, each providing for a series of bridges to be considered as a unit and to be financed by the issuance of bonds, which, in turn, are to be retired by the income from tolls.

The bridges in the first group are needed and the tolls will easily finance their construction and maintenance. The bridges of the second group are in isolated areas and on secondary highways and it is doubtful whether any of them will prove paying investments. As a whole, this series will be difficult to finance from the tolls collected. The acts authorizing these two groups also fixed the charges that will be made, thus fixing the income which the Highway Department may reasonably expect from the structures. Had studies of income been made before the legislation was enacted, several of the structures would not have been authorized because it would be cheaper to operate ferries without charge. The bridges of the third series will serve a real traffic need and at least two of them will finance themselves. It is probable that the two profitable bridges will finance the third.

All the bridges in each group were classed together, and were to be paid for out of the tolls, the group to be made free as soon as the bonds on the whole series were retired.

Estimate of Traffic.—The State Highway Department of Tennessee has been conducting studies for a period of years in order to determine the

* Discussion of the paper by G. F. Schlesinger, M. Am. Soc. C. E., continued from December, 1928, *Proceedings*.

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‡ Received by the Secretary, January 16, 1929.

relation between the average flow and the seasonal flow of traffic as well as the volume of traffic that may be reasonably expected over the proposed bridges. The data show that the traffic in August is approximately 33% more than the average for the year. The total 24-hour traffic is 133% of the daylight flow. This was determined by taking night counts. A 12-hour count taken in August, therefore, approximates the average 24-hour flow throughout the year. In 1926 a general count was made throughout the State, and this is used as a basis for predicting future travel.

In Tennessee, as in other States, the increase in traffic flow has been approximately proportional to that of motor vehicle registration. This has been used as a basis of foretelling the volume of future traffic. Predictions made about 1923 have agreed fairly well with the actual registration for 1926 and 1927.

A more accurate value may be determined if the many sources are estimated separately. A study of vehicle license numbers from the several sections of the State has shown that the preponderance of travel is from the county in which the road is located or from the adjoining counties. The study was divided, therefore, into: (*a*) Local travel; (*b*) travel from counties of large registration; (*c*) travel from other counties of the State; and (*d*) travel from other States. No effort is made to separate the truck flow from the travel of automobiles, on the assumption that the same law applies for the trucks as for the passenger car.

Local Travel.—To estimate local travel, a large number of stations were selected at which the flow of vehicles was obstructed as little as possible. It is well known that detours, bad stretches of road, construction under way, and other obstructions, limit the free flow of vehicles. From preceding counts an estimate was made of the flow of local vehicles at the stations. From records of motor-vehicle registration, an estimate was made of tributary motor vehicles by assuming that they would pass the station in proportion to their remoteness considering only the counties in tributary areas. In other words, of the total cars registered in the various counties, a certain percentage will contribute potentially to the daily traffic density at a fixed observation station. The writer has estimated this percentage as some ratio of the relative distances or remoteness from the observation station.

A typical analysis is shown in Table 10 for the station at Spring Hill, Tenn., near the Williamson-Maury County line. The percentages in Column (2) varied with the distance from Spring Hill and from a knowledge of the topography in the area.

The local travel at Spring Hill in 1926 was 573 vehicles per 12-hour day, which was a ratio of travel to weighted registration of 0.091. If two main thoroughfares are parallel across a county, certain vehicles will be tributary to one highway and others will flow over the other highway. About twenty stations were studied in a similar way, and it was found that the weighted motor-vehicle registration was ten times the local travel at the stations.

It is assumed, therefore, that the travel at other unobstructed stations will be approximately one-tenth the tributary motor-vehicle registration (see Table 11). An estimate was then made of the motor-vehicle registration tributary to all the proposed bridge sites. This was done by projecting the curve of regis-

tration from 1920, through 1927, to 1940. Many factors cause the registration to change, but all of them can hardly be estimated in advance. The trend of the preceding seven years was probably the best information at hand and was therefore used in the study. The population was estimated from the Census reports and the registration was taken from the records of the Department of Finance and Taxation through 1927.

TABLE 10.—ESTIMATED NUMBER OF REGISTERED VEHICLES THAT CONTRIBUTE TO TRAFFIC DENSITY AT SPRING HILL STATION.

County. (1)	Percentage of total cars registered that contribute to traffic. (2)	Cars registered in county. (3)	Weighted values. (4)
Williamson	80	2 908	2 326
Maury	60	3 164	1 898
Giles	30	2 655	797
Lawrence	30	2 315	695
Marshall	20	1 917	283
Hickman	10	1 115	112
Lewis	10	644	64
Total	....	.....	6 175

The weighted value taken is that obtained by assuming that the percentage of the cars tributary to the station varied with the distance from the source to the station. Some of the percentages are somewhat arbitrary and were determined from a personal knowledge of the topography and characteristics of the sites.

TABLE 11.—ESTIMATED RATIO BETWEEN NUMBER OF CARS PASSING A STATION AND TOTAL CARS REGISTERED IN THE DISTRICT SERVED.

Route.	Location.	Cars registered.	Number of cars counted.	Ratio.
1	Blountville	6 900	692	0.100
24	Overtown County Line	2 150	210	0.097
1	West of Crossville	2 130	230	0.108
6	Spring Hill	6 275	573	0.091
3	Ripley	6 220	685	0.110
28	Dunlop	1 340	149	0.104
16	East of Shelbyville	6 125	613	0.099
64	One mile south of Madisonville	1 200	60	0.050
34	Seven miles west of Morristown	3 400	350	0.103
1	Church Hill	4 300	500	0.125
10	Ten miles south of Murfreesboro	4 160	500	0.120
52	Macon County	2 400	280	0.115
1	McEwen	3 400	350	0.103
6	Near Franklin	5 500	600	0.110
76	Seven miles south of Clarksville	2 300	175	0.075
3	South of Union City	4 500	360	0.080
42	Modina	5 000	360	0.072
1	Huntingdon	7 000	610	0.087
Totals, or average.		74 360	7 289	0.097

A typical analysis is given in Table 12 for Route 15 at the bridge over the Tennessee River between Waynesboro and Selmer, Tenn. (Bridge 3, Fig. 11).

TABLE 12.—TYPICAL ANALYSIS OF POPULATION TRAFFIC AT THE BRIDGE AT SAVANNAH, HARDIN COUNTY, TENN.

County.	Percentage of total registration effective.	Population.		Estimated Population That Contributes to Traffic at the Savannah Bridge.	
		1910.	1920.	1910.	1920.
Hardin	90	17 521	17 291	15 769	15 562
McNairy	70	16 356	18 350	11 449	12 845
Wayne	50	12 062	12 877	6 031	6 439
Lawrence	10	17 569	23 593	1 757	2 359
Chester	10	9 090	9 669	909	967
Hardeman	10	23 011	22 278	2 301	2 228
Total	....	.....	.....	38 216	40 400
Increase in 10 years	....	.....	.....	.....	2 184

Weighted registrations were obtained in a similar manner, using the actual registrations for each county each year (see Table 13).

TABLE 13.—WEIGHTED REGISTRATIONS FOR STUDY OF SAVANNAH BRIDGE.

Year.	1910.	1920.	1921.	1922.	1923.	1924.	1925.	1926.	1227.	1930.	1935.	1940.
Population	38 216	40 400	40 618	40 836	41 054	41 272	41 490	41 708	41 928	42 580	43 670	44 760
Number of cars registered	.....	756	837	892	930	1 465	2 030	2 638	2 828	3 400	3 850	4 000
Persons per vehicle	.....	52.5	48.5	45.5	44.0	28.0	20.3	15.9	14.8	12.5	11.6	11.2

The local unobstructed travel, therefore, is estimated at 340 vehicles in 1930, 385 in 1935, and 400 in 1940.

Large Cities.—The four large counties contain 48.5% of the motor vehicles registered in the State. It is desirable, therefore, to study the effect of motor-vehicle concentration in the large cities. Fortunately, information was available for this study. During the 1926 count, automobile license numbers were listed at more than forty stations. From the records in the Bureau of Registration, the location of ownership was determined and a tabulation was made for Davidson, Knox, Hamilton, and Shelby Counties, which include the Cities of Nashville, Knoxville, Chattanooga, and Memphis, respectively. Fig. 11 (*a*) shows by contour the number of cars per day originating in Davison County as they traveled throughout the State of Tennessee in 1926. The composite map (Fig. 11 (*b*)) shows the effect of nearly 50% of the automobiles of the State on the travel at the several bridge sites.

By projecting the registration for the four large counties forward to 1930 and 1940, an estimate may be made of the probable travel over the several bridges from this source.

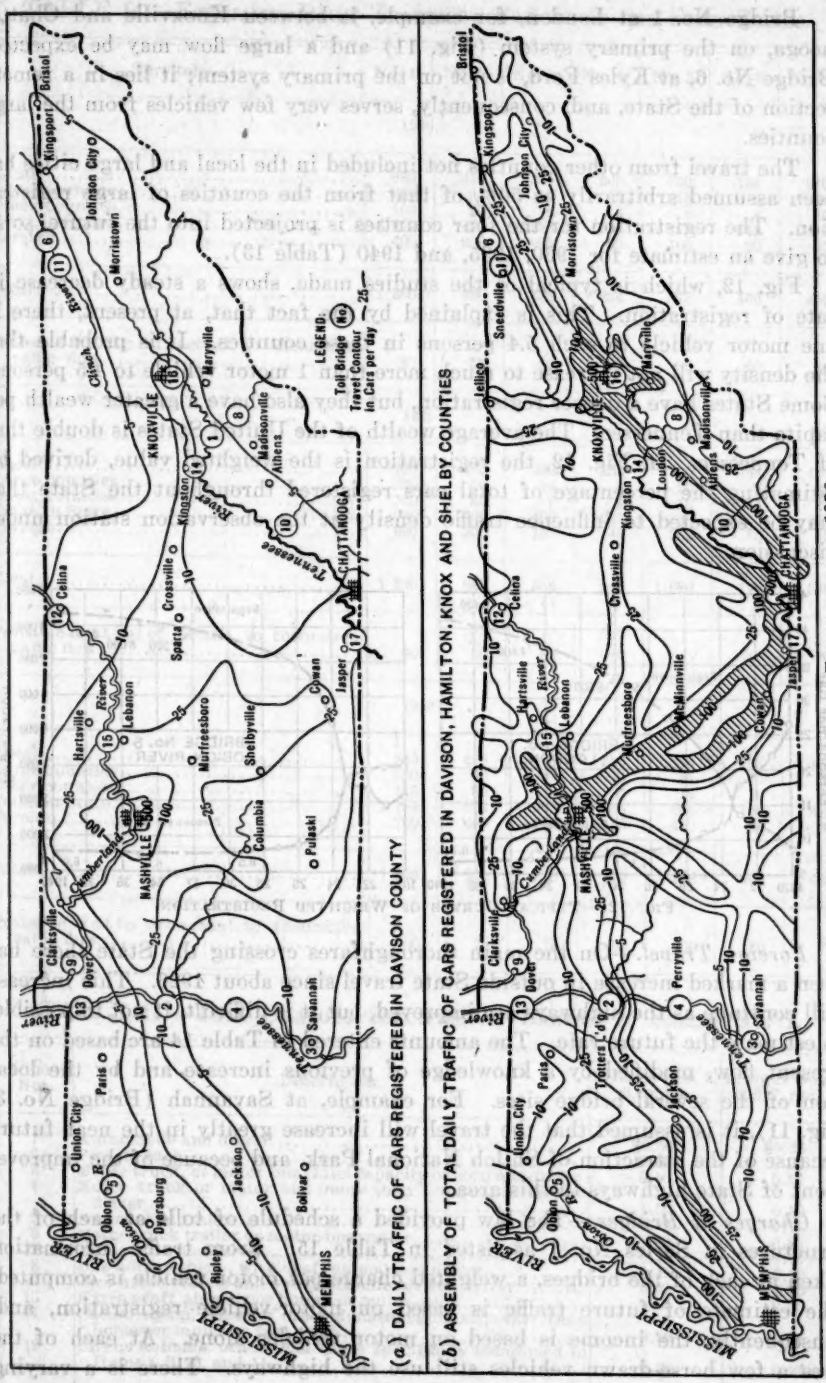

FIG. 11.—TRAFFIC MAP OF TENNESSEE FOR 1926.

Bridge No. 1 at Loudon, for example, is between Knoxville and Chattanooga, on the primary system (Fig. 11) and a large flow may be expected. Bridge No. 6, at Kyles Ford, is not on the primary system; it lies in a remote section of the State, and, consequently, serves very few vehicles from the large counties.

The travel from other counties not included in the local and large cities has been assumed arbitrarily as 50% of that from the counties of large registration. The registration for the four counties is projected into the future, so as to give an estimate for 1930, 1935, and 1940 (Table 13).

Fig. 12, which is typical of the studies made, shows a steady decrease in rate of registration. This is explained by the fact that, at present, there is one motor vehicle to each 5.4 persons in these counties. It is probable that the density will not increase to much more than 1 motor vehicle to 4.5 persons. Some States have a denser registration, but they also have a greater wealth per capita than Tennessee. The average wealth of the United States is double that of Tennessee. In Fig. 12, the registration is the weighted value, derived by estimating the percentage of total cars registered throughout the State that may be expected to influence traffic density at the observation station under discussion.

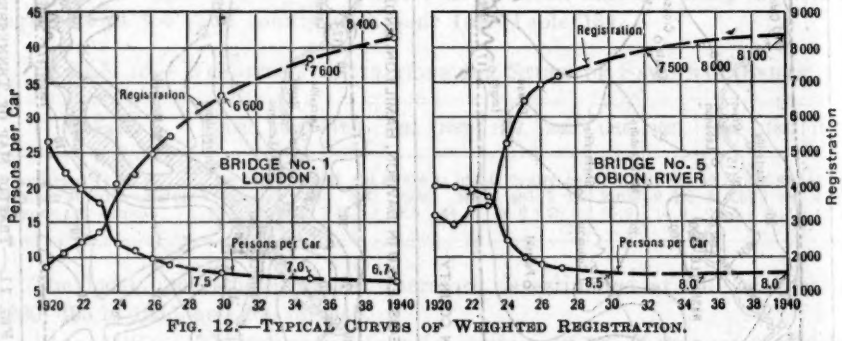

FIG. 12.—TYPICAL CURVES OF WEIGHTED REGISTRATION.

Foreign Travel.—On the main thoroughfares crossing the State there has been a marked increase in outside State travel since about 1926. This increase will continue as the highways are improved, but it is difficult, if not impossible, to estimate the future rate. The amounts entered in Table 14 are based on the present flow, modified by a knowledge of previous increase and by the location of the several bridge sites. For example, at Savannah (Bridge No. 3, Fig. 11), it is assumed that the travel will increase greatly in the near future because of the attraction of Shiloh National Park, and because of the improvement of State highways in this area.

Charges at Bridges.—The law provided a schedule of tolls at each of the structures in Series No. 1 as listed in Table 15. From traffic information taken at each of the bridges, a weighted charge per motor vehicle is computed. The estimate of future traffic is based on motor-vehicle registration, and, consequently, the income is based on motor vehicles alone. At each of the sites a few horse-drawn vehicles still use the highways. There is a varying

TABLE 14.—ESTIMATED DAILY TRAVEL ON BRIDGES OF THE FIRST SERIES.

Territory.	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 8.
1930							
Local	660	415	340	205	750	134	370
Large counties	200	45	12	6	15	6	30
Other counties	100	20	6	3	8	3	15
Total, in State	960	480	358	214	773	143	415
Foreign	400	200	75	20	100	20	35
Total	1 360	680	433	234	873	163	450
Probable ratio of actual to theoretical traffic flow	80	55	50	35	90	30	60
1935							
Local	760	450	385	235	800	148	420
Large counties	220	50	14	8	20	8	35
Other counties	110	25	7	4	10	4	20
Total, in State	1 090	525	406	247	830	160	475
Foreign	600	300	150	40	250	30	60
Total	1 690	825	556	287	1 080	190	535
Probable ratio of actual to theoretical traffic flow	90	60	60	40		40	70
1940							
Local	840	480	400	250	810	153	480
Large counties	230	60	15	10	25	10	40
Other counties	115	30	8	5	13	5	20
Total, in State	1 185	570	423	265	848	168	540
Foreign	750	400	250	50	400	40	80
Total	1 935	970	673	315	1 248	208	620
Probable ratio of actual to theoretical traffic flow	100	70	75	50	100	50	75

TABLE 15.—TOLL CHARGES SPECIFIED ON BRIDGES IN SERIES NO. 1.

Item No.	Description.	Bridges Nos. 2, 3, and 4.	Bridges Nos. 1, 5, 6, 7, and 8.
1	Automobile and driver	$0.50	$0.25
2	Person, each	0.05	0.05
3	Motor truck, or motor bus (1 ton capacity or less) and driver	0.50	0.25
4	Motor truck or motor bus (more than 1-ton capacity) and driver	1.00	0.50
5	Automobile trailer	0.25	0.15
6	Motor-truck trailer or motor-bus trailer	0.50	0.25
7	Motorcycle and driver	0.25	0.15
8	One-horse vehicle, with draft animal and driver	0.15	0.10
9	Two-horse vehicle and two draft animals and driver	0.25	0.15
10	Extra draft animals or horses or mules, each	0.05	0.05
11	Cattle, sheep, hogs, or other domestic animals, other than horses or mules on foot, each	0.02	0.02
12	Circus animals, other than those specifically mentioned in Items 8, 9, 10, and 11	0.25	0.25

distribution of light and heavy trucks, depending on the location. At each bridge an estimate of income has been made from actual travel, according to the schedule of the tolls listed in Table 15, and from this estimate the average charge per motor vehicle is made. The Tennessee cars are kept separate from the foreign cars because the percentage of trucks from outside the State is very small. The average charge for Tennessee motor vehicles is obtained by dividing the total receipts for the day by the total motor vehicles passing the station. The charge for foreign automobiles is obtained by adding to the amount received for the automobiles, one and one-half times 5 cents; making the charge for Bridges Nos. 2, 3, and 4, equal to 57.5 cents; and for Bridges Nos. 1, 5, 6, and 8, a charge of 32.5 cents. A sample computation is given for the Loudon Bridge (No. 1, Fig. 11), as follows:

Automobiles	$640 \times \$0.25 =$	\$160.00
Passengers	$1300 \times 0.05 =$	65.00
Light trucks	$32 \times 0.25 =$	8.00
Heavy trucks	$35 \times 0.50 =$	17.50
Trailers	$1 \times 0.25 =$	0.25
Horse-drawn	$1 \times 0.15 =$	0.15
Horse-drawn	$5 \times 0.10 =$	0.50
$640 + 32 + 35 = 707$ vehicles $=$		\$251.40
$707 \times 35.5 + =$		251.40
$\$0.25 + 5 \times \$0.015 =$		\$0.325

The average number of passengers per car in Tennessee, as shown by previous counts, is 2.5 persons per car. Data taken at the ferries indicate a similar number of passengers. An automobile, therefore, will pay 25 cents for driver and car and 5 cents additional for each passenger.

To obtain the daily estimated tolls for 1930, 1935, and 1940, Tennessee travel is multiplied by the average charge for each station, and the foreign travel is multiplied by 32.5 cents, or 57.5 cents, according to the schedule of tolls fixed by law. The weighted charges for each bridge in Group 1 are listed in Table 16.

TABLE 16.—Weighted Toll Charges for Bridges in Group 1.

Description.	Charge per vehicle.						
Bridge No.	1	2	3	4	5	6	8
Local	\$0.355	\$0.650	\$0.575	\$0.610	\$0.340	\$0.330	\$0.325
Foreign	\$0.325	\$0.575	\$0.575	\$0.575	\$0.325	\$0.325	\$0.325

Expectancy.—In making an estimate of the travel at the several bridges, free flow was assumed from all the tributary area. This assumption is far from the truth for many of the bridges proposed. Some of the locations are in areas where highways will have to be constructed. The estimates made

from the curves, therefore, are modified as nearly as may be to suit the station in question.

TABLE 17.—PROBABLE LOCAL TRAFFIC FLOW OVER BRIDGES IN THE FIRST SERIES.

Bridge No.*	Traffic, 1927.	Estimated traffic, 1927.	Expectancy, in percentage of theoretical or estimated traffic, 1927.
1	717	1 000	72
2	210	425	50
3	120	300	40
4	40	150	26
5	600	750	80
6	25	125	20
8	155	325	48

* See Fig. 11.

Traffic was counted in August, 1927, at all the stations. An estimate has been made of the probable local flow from the motor-vehicle registration. Table 17 gives the actual and estimated travel for 1927 and shows the percentage that the actual travel is of the theoretical. From Table 19 and a knowledge of the particular locations and the progress of highway improvement it is possible to derive the percentage expectancy for the several sites.

TABLE 18.—ESTIMATED DAILY AND ANNUAL TOLLS FROM BRIDGES IN THE FIRST SERIES.

Bridge	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 8.
1930							
Tolls per day:							
Tennessee cars	$ 339	$ 312	$ 205	$ 130	$ 263	$ 47	$ 135
Foreign cars	130	115	43	11	32	6	11
Total per day	469	427	248	141	295	53	146
Percentage of expectancy	80	55	50	35	90	30	60
Probable daily	$ 374	$ 235	$ 124	$ 49	$ 265	$ 16	$ 88
Annual	136 000	85 500	45 200	18 000	96 500	5 800	32 000
1935							
Tolls per day:							
Tennessee cars	$ 386	$ 311	$ 234	$ 150	$ 282	$ 53	$ 154
Foreign cars	195	172	86	23	81	10	19
Total per day	581	513	320	173	363	63	173
Percentage of expectancy	90	60	60	40	95	35	70
Probable daily	$ 523	$ 308	$ 192	$ 69	$ 344	$ 22	$ 121
Annual	191 000	112 000	70 000	25 200	125 000	8 000	44 000
1940							
Tolls per day:							
Tennessee cars	$ 420	$ 370	$ 243	$ 162	$ 288	$ 55	$ 175
Foreign cars	243	230	143	29	130	13	26
Total per day	663	600	386	191	418	68	201
Percentage of expectancy	100	70	75	50	100	40	75
Probable daily	$ 663	$ 420	$ 290	$ 95	$ 418	$ 27	$ 150
Annual	242 000	153 000	106 000	34 800	152 000	9 900	55 000

Expected Income.—Having tabulated the probable travel for 1930, 1935, and 1940, it only remains to compute the income for each year modified by the expectancy as listed in Table 14. The specified toll charges are given in Table 15 and simple multiplication therefore produces the result shown in Table 18.

TABLE 19.—ESTIMATED ANNUAL COST AND TIME REQUIRED FOR RETIREMENT OF BONDS ON BRIDGES OF THE FIRST SERIES.

Bridge No.	Income.			Cost.	Interest at 4½ per cent.	Upkeep and operation.	Annual cost	Time to retire cost, in years.
	1930.	1935.	1940.					
1	$136 000	$190 000	$242 000	$ 800 000*	$36 000	$10 000	$ 46 000	7
2	85 500	112 000	153 000	1 014 359	45 600	10 000	55 600	13
3	45 200	70 000	106 000	850 513	38 300	10 000	48 300	15
4	18 000	25 200	34 800	727 445	32 800	10 000	42 800	Indefinite
5	96 500	125 000	152 000	214 323	9 600	9 000	18 600	3
6	5 800	8 000	9 960	62 104	2 800	5 000	7 800	25
8	32 000	44 000	55 000	155 481	7 000	5 000	12 000	6
Total...	$419 000	$575 200	$752 700	$3 824 225	$172 100	$59 000	$231 100	11

* Estimated cost.

All the bridges of this group have been let to contract. The costs for each structure and the cost for the whole group, therefore, are approximately known. The probable annual cost may be easily estimated. Table 19 gives the estimated annual cost and the probable period required to retire the cost of construction.

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PAPERS AND DISCUSSIONS

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WATER SUPPLY PROBLEMS OF A DESERT REGION

Discussion*

By Lewis R. East, Assoc. M. Am. Soc. C. E.

Lewis R. East,† Assoc. M. Am. Soc. C. E. (by letter).‡—Australia has a land area of 3 000 000 sq. miles, which is slightly greater than that of the United States. It is, however, less favored in the matter of rainfall and, in consequence, is peculiarly destitute of great rivers. Fortunately, over large areas, underground water supplies, of good quality, can be obtained at reasonable depths. In the arid and semi-arid regions, considerable work has been done in providing water supplies for domestic, stock, and irrigation purposes.

Apart from comparatively short coastal streams, the only rivers of consequence are the Murray and its tributaries, which, during the winter months, have a total navigable length of approximately 3 212 miles. The Murray Basin has an area of 414 000 sq. miles, of which only 159 000 sq. miles can be regarded as contributing to the river flow, and some of this, to a small degree only.

Part of the catchment has a heavy rainfall, and some of the mountainous areas are covered with snow for several months in each year. The average annual rainfall over the whole area is, however, only 13 in.

Evaporation losses appear, from observations, to range up to a maximum of about 60 in. per annum. In a year of low rainfall, the total discharge of the Murray River System has fallen as low as 2 327 000 acre-ft., while in a wet year the run-off is more than twelve times this amount, the average being 9 328 000 acre-ft.

It has long been realized that storages are essential for the economic use of available supplies, and—especially in Victoria and New South Wales—all political parties for many years have followed progressive policies in regard to water conservation.

* This discussion (of the paper by William E. Rudolph, M. Am. Soc. C. E., published in October, 1928, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, February 13, 1929.

On the Murrumbidgee River, one of the tributaries of the Murray, a cyclopean concrete dam 200 ft. high was completed in 1925 to impound 772 000 acre-ft. of water, and on the Upper Murray good progress is being made with the Hume Reservoir which, when completed, will store 2 000 000 acre-ft. This storage will rank with Gouin, Elephant Butte, and La Boquilla among the world's largest artificial storages. On the Goulburn River and other tributaries storages have been completed with quantities totaling 800 000 acre-ft., and other works are under construction which will bring the total storage on the Murray and its tributaries to more than 4 000 000 acre-ft.

The waters thus conserved on the Murray and other smaller river systems in Victoria are being used largely for irrigation; but they will also be used for domestic and stock supplies on several millions of acres outside the irrigation areas.

The State of Victoria under the administration of the State Rivers and Water Supply Commission (corresponding to the Bureau of Reclamation, of the United States) already has made considerable progress in water supply and irrigation development. It should be remembered that one of the first irrigation plans in Australia—and certainly the earliest, involving high-lift pumping on a large scale—was inaugurated in 1887 by Mr. George Chaffey who later played such an important part in the development of the Imperial Valley, California.

In this discussion, the writer wishes to describe some of the problems of domestic and stock water supply in arid and semi-arid parts of Australia, and the methods by which these problems have been solved. This will be confined to a description of the various water-works supplying the northwest part of the State of Victoria, in Southeastern Australia, over which the mean annual rainfall varies from 20 in. in the more favored Wimmera Area to 12 in. in the far north, or Mallee Area.

Geology.—The geology of the Mallee Area has an important bearing on its water supply. In geologically recent times it was covered by a great inland bay into which discharged the head-waters of what are now the rivers of the Murray System. Later, as this bay became entirely cut off from the sea by drifting sands and marsh growths, lacustrine conditions developed, and the depression was gradually filled with sands and gravels brought down from the surrounding ranges. The whole area thus became a vast sandy plain over the margins of which the streams continued to pour their burdens of silts, sands, and gravels.

The materials laid down before and after the area was raised are entirely distinctive—the latter, or fluviatile, being gray, and the former, or lacustrine, being red. While the fluviatile deposits were gradually extending over the edges of the lacustrine area, the surface of the remainder, under semi-arid conditions, was being blown broadcast by wind action until the whole of the exposed lacustrine area had been redistributed. The Mallee country now consists of low sand ridges interspersed by harder flats, generally with an east and west tendency. The larger streams have joined to become the Murray System and have cut their way through these deposits exposing cliffs varying from 60 to 180 ft. in height. The lesser streams from the south "lose them-

selves" in the sands and do not reach the Murray. Many salt lakes remain in low-lying sections, but throughout the Mallee (the Victorian Area is 11 000 000 acres), there are no defined watercourses other than the Murray Canyon itself.

Natural Water Supplies.—As may be expected, natural surface water supplies are exceedingly scarce and unreliable in the Mallee country, which up to comparatively recent years was shown on maps as "sandy desert covered with dense scrubs of *Eucalyptus Dumosa*". This *Eucalyptus Dumosa* is the Mallee scrub which gives its name to the area. After heavy rains, surface water can frequently be obtained where, under favorable conditions, rain water collects and lies for months in depressions or clay pans. "Soaks" can also be located by experienced observers so that ground-water can be obtained by digging a few feet. The aborigines were adepts at this, and many native wells have been found. Mallee exploration was attended by great hardships due to lack of water; one of the early surveyors reached the river only after killing his horse and drinking its blood.

The first efforts of early settlers were naturally to develop available "soaks" and catchments. Where water collected naturally at clay pans, a hole was sunk and lined with pine logs (a native tree, *Callitris Robusta*, which is not the true pine). This was roofed over to reduce evaporation and it held small quantities of water for domestic use over long periods of dry weather.

Shafts or Wells.—In certain localities near the mountains, shafts were sunk to underground water—sometimes to depths of several hundred feet in extreme cases—and timbered. Water was raised by means of horse works, windmills, or engines.

Catchment Tanks.—Catchments are sometimes natural, that is, open spaces with hard ground; but generally they are prepared by clearing suitable slopes and plowing complete systems of catch-drains leading to excavated tanks which must be located in good "holding ground", that is, in clay. Catchment tanks vary in depth according to the depth of clay available, and excavation must not go through the clay to underlying porous drifts. Side slopes vary from 1 on 2 to 1 on 3, and for average depths, up to 15 ft., the cost of excavation by scoop teams is about $0.30 (1s 3d) per cu. yd.

It has been found that, with a 12-in. rainfall, approximately 20 acres of catchment are required for each 1 000 cu. yd. of storage (170 000 Imperial gal.). The actual percentage of run-off is very low, because a 0.7-in. (50-point) catchment is rare, that is, a catchment which will run water with a 0.7-in. rain. Practically all supplies are obtained from occasional heavy thunderstorms of more than about 1 in. For grazing stock to trample the catchment is advantageous, but cultivation, of course, cannot be permitted. The minimum storage capacity for a wheat farm of about 1 sq. mile in area, is about 2 000 cu. yd. which requires about 40 acres of catchment. The best catchments are reserved for public tanks and the largest tank thus far constructed and filled in the Mallee has a capacity of 40 000 cu. yd. When Mallee lands are thrown open for selectors, an addition of from $0.73 to $0.97 (3s to 4s) per acre is made to the purchase price to pay for preliminary road and water supply works. This addition or "loading" is paid by the settlers at the same time as land installments over a period of from 20 to 40 years. The full amount of

the loading is, therefore, not available immediately for works; but the present value—approximately three-fifths—is spent in constructing public catchment tanks and in grubbing roads for access to the farms.

To reduce evaporation losses which are considerable, tanks are sometimes covered by roofing with wire netting overlaid with dry grass tussocks to a depth of from 1 to 2 ft. Usually, only the edges are covered, the center being left open. This is found to be quite effective.

Rock Catchments.—Mention should be made of the "rock catchments" of Western Australia where outcrops of solid granite have been surrounded by cement masonry drains leading to adjoining excavated storages. There are catchments of as much as several hundreds of acres in area and 150 ft. above the general level of the surrounding country. These rock catchments yield as much as 95% of the first inch of rain and practically all of any rain following immediately.

Storages are generally excavated to a depth of about 15 ft., the bottom 10 to 12 ft. being water-tight clay, above which there may be from 3 to 5 ft. of porous material. Loss of water is prevented by excavating a trench to clay under the site of the spoil-banks and filling this with clay pug.

Bores.—Below the æolian surface soils of the Mallee and red lacustrines there is a more or less impervious blue clay band under which are considerable thicknesses of marine tertiaries, containing water under artesian conditions. This water is invariably mineralized to a greater or less extent—sometimes approaching saturation—and the minerals in solution are as invariably in the proportions of sea water. There are generally several layers of water at different depths, that is, perched basins, and the top water is usually of very inferior quality. In the southwestern part of the Mallee adjoining the South Australian boundary, usable water containing about 70 parts of dissolved salts per 100 000 is obtained by boring to depths of from 100 to 200 ft., which rises to within about 50 ft. of the surface. At greater depths of from 450 to 550 ft., water containing as little as 35 parts of dissolved solids per 100 000 has been obtained, and as much as 6 000 gal. per hour has been pumped from a 6-in. bore at this depth without unduly lowering the level of water in the bore. To the east of this district the depth to usable water increases and the quality of the water deteriorates until in about 60 miles a limit of about 500 parts per 100 000 is reached. Farther east the quality is even worse.

Limits of salinity (total dissolved solids) vary according to the nature and proportions of the salts in solution, but in the Victorian Mallee (sea-water proportions) may be taken as follows:

Human consumption..	Good.....	30	parts	per	100 000	;	0.05	oz.	per gal.
	Medium ..	60	"	"	"	;	0.10	"	" "
	Bad.....	150	"	"	"	;	0.25	"	" "
	Limit	620	"	"	"	;	1.00	"	" "
Working horses	Good.....	300	"	"	"	;	0.50	"	" "
	Fair.....	400	"	"	"	;	0.625	"	" "
	Limit	500	"	"	"	;	0.80	"	" "
Grazing stock	Limit	900	"	"	"	;	1.50	"	" "

Under inland conditions sheep have been known to do well on water containing 2 oz. to the gallon when they drank it continuously.

The area under which usable water can be obtained covers about 1 000 000 acres of agricultural lands. In this region the State Rivers and Water Supply Commission has driven nearly 100 bores, each supplying water for eight or nine farm holdings of 700 acres. The bores (mostly 5 in. in diameter) are cased so as to cut off waters of inferior quality, and, in general, are equipped with windmills 12 ft. in diameter on 30-ft. steel towers. Storage is provided by 4 000-gal., corrugated-iron tanks on timber or steel stands. The cost of a bore complete with mill and tank averages about $1 200 (£250), the actual boring costing about $2.43 (10s) per ft.

As the water is generally neutral or slightly alkaline, the casing has a long life; no failures due to its corrosion are known in this area although many of the bores have been operating for more than fifteen years. The whole cost of constructing and maintaining the bores is met by rating the lands that are benefitted; the actual rate averages about $0.04 (2d) per acre per annum. The cost of maintenance and repairs is about $100 (£20) per bore per year.

Channels.—Gravitation Supplies.—The bore area is limited. Catchment tanks, which completely failed in times of drought, proved inadequate to meet the demands made on them as the country developed and the urgent necessity of providing assured and ample water supplies for working and grazing stock has resulted in the development of an extraordinarily extensive system of channels bringing water from the wet south to the dry north.

As might be anticipated from its geological formation the Wimmera-Mallee Area has a general northerly slope from the southern water-shed—the Grampians and the Dividing Range. From these ranges a number of small rivers cross the Wimmera plains and disappear in the sandy soils of the Mallee. None of them reaches the Murray or the sea, even in times of great flood. As early as 1880, the natural flow of the chief of these streams, the Wimmera River, was diverted into channels by means of timber weirs and carried to excavated tanks on farms in sections of the country where there were no defined watercourses. By 1891 about 500 miles of these channels had been constructed. The only storage at this time was Wartook Reservoir of 24 000 acre-ft. capacity. A further storage (Lake Lonsdale, 45 000 acre-ft.) was completed in 1902 at the end of a drought year in which the whole distribution system practically failed. After the constitution of the State Rivers and Water Supply Commission in 1906, all these works came under Government control and considerable extensions were made.

The old method of using creeks and water-courses as main channels was a wasteful method which resulted in excessive evaporation and seepage losses. The Commission abandoned all natural depressions and laid out a system of main channels suitably located along main ridges to command practically the whole of the Wimmera and Mallee Districts. Water from storages has been taken across rivers by means of low-level weirs or diversion dams which not only enables it to cross the streams, but also permits any river flow available to be diverted into the main channel. In normal seasons the river flow is sufficient to keep the main channel full throughout the winter and spring without drawing on the storages which are thus kept in reserve to make up deficiencies during periods of little or no river flow.

Since 1906 the area served by the layout, known as the Wimmera-Mallee Gravitation Scheme, has been more than trebled until at present about 11 000 sq. miles are supplied from its channels. To serve this area, which includes 35 townships and has a population of 100 000, more than 5 100 miles of channels have been constructed by the Commission. The longest Government channel carries water a distance of 300 miles from the storages in the hills into the far north. This great extension of the original plan has necessitated full development of the catchments of all streams supplying the area and the total capacity of storages is now 200 000 acre-ft. Slightly less than one-half of this is required for watering the whole area in an average year.

The channels vary in capacity from 700 to 4 cu. ft. per sec. and their grades also vary from 10 ft. per mile to 9 in., depending on the fall of the country and the size of the channels. The velocity of flow is usually kept about 0.5 ft. per sec. Channels in cuts as deep as 4 ft. have side slopes of 1 on 1 with 1 on 1.5 for deeper cuts. In very sandy land, such as that in desert country, the channels are lined with about 1 ft. of clay or loam to minimize seepage losses, but in general they are merely excavated in the earth. In designing channels of this nature it is usual to allow 1 cu. ft. per sec. for every 40 sq. miles, but no main channel should be designed for less than 10 cu. ft. per sec. and the smallest distributary (bed width usually 3 ft. for reasons of construction), should be not less than 4 cu. ft. per sec. The extreme length of a small channel of this size should not exceed 10 to 15 miles. For longer channels the capacities must be increased, regardless of the actual area served. It must be remembered that no irrigation is permitted from the channels of this system; all the water available is required to run the channels, in which seepage and evaporation losses are very great, and to fill excavated storages on each farm once per year, with enough water to serve all needs for the full twelve months until the next run of the channels. As far as possible the channels are run during the winter months in order to reduce seepage and evaporation losses. Farmers are required to provide tanks on their holdings of a total capacity equivalent approximately to 4 cu. yd. for each acre; that is, 2 500 cu. yd. for a 640-acre holding. The minimum depth required is 12 ft. and, as channels are generally on high ground, storage above ground level is frequently possible. Excavation is, therefore, often only two-thirds of the full capacity, the remainder being carried above ground level by banks. The minimum supply necessary for an average farm of about 640 acres is about 750 cu. yd., or 120 000 Imperial gal. per year. This will suffice for about 200 sheep, 6 cattle, 12 horses, and 10 persons. An open storage of 2 500 cu. yd., or 425 000 Imperial gal., will yield this quantity after allowing for all evaporation losses and waste during the year.

As in the bore areas, the landholders are rated on a valuation basis to cover the cost of the project. The average rate is from $58 to $100 (£12 to £20) per sq. mile, with a minimum of about $52 (£10/13/4) per sq. mile, or $0.08 (4d) per acre. The charge is not based on the quantity of water supplied. To encourage the landholders to put down storages of ample capacity the Commission charges no more for filling a 15 000-cu. yd. tank

than for one of 2 500 cu. yd. Under favorable conditions 200 miles of channel can be run in 30 days, but in more porous country the time required might be 60 days.

Maintenance consists largely in removing sand-drift blown into channels during hot windy weather. In the parts of the Mallee particularly liable to "drift", channels are occasionally entirely filled with sand blown by one storm. Thirty-five townships within the area commanded by the Wimmera-Mallee System have pipe reticulations supplied from local storages which are filled annually from its channels.

The project has cost $8 750 000 (£1 800 000) to date, or $0.97 (4s) per acre, and is proving a sound financial proposition to the State apart from its general contribution to the prosperity of the country.

Channels.—Pumped Supplies.—In addition to this great gravitation system the Commission has completed several independent projects by which large areas are supplied with water for domestic and stock purposes through channels filled by pumping from the River Murray. In the County of Millewa approximately 1 000 sq. miles are thus supplied through separate low-level and high-level channel systems. Three pumping plants are required to serve the lower system, which is approximately 145 ft. above river level. The main lift is 113 ft., and the minor lifts, 13 ft. and 19 ft., respectively. The high system requires a further lift of 135 ft., thus raising the water a total of 280 ft.

For the low-level system (known as the Lower Millewa Water-Works District), the average rate is $0.16 (8d) per acre, or $103 (£21/6/8) per sq. mile. For the high-level (or Upper Millewa District), the rate is $0.20 (10d) per acre, or $129 (£26/13/4) per sq. mile. The length of public channels constructed in the Millewa Districts to date is approximately 700 miles.

Pipe Reticulation.—In the State of Victoria the supply of water to farms through pipe reticulations has not been extensively adopted owing to its great cost. However, in the adjoining State of South Australia several such layouts have been in operation for a number of years and one, the Tod River Water Scheme* when completed, will have in its 240-mile main one of the longest pipe lines in the world, second only to the 350-mile Kalgoorlie Goldfields Main in Western Australia.

The Tod River Water Scheme is designed to supply water to 10 000 sq. miles of agricultural land through a 240-mile main and 1 000 miles of distributaries. Water from the Tod River and its tributaries is diverted into a reservoir of approximately 10 000 acre-ft. capacity and thence pumped through 2½ miles of 18-in. rising main to a small concrete lined reservoir on Knotts Hill, holding 20 acre-ft. From this reservoir the whole area is commanded by gravitation. The main varies from 20 to 10 in. and is of steel—the last 110 miles being concrete-lined. A break-pressure basin, also of 20 acre-ft. capacity, is provided at about 128 miles from the beginning of the main and 259 ft. below the full supply level of the Knotts Hill Reservoir.

* *The Engineer*, August 10, 1928, Vol. CXLVI, p. 149.

Rating is based on the unimproved valuation and varies from \$0.08 (4d) to \$0.14 (7d) per acre on all land within 1 mile of a main. The estimated consumption of water, based on actual figures in other areas is 132 gal. per acre, or 84 480 gal. (Imperial) per sq. mile. For the rate, 43 000 gal. are allowed and the excess is charged for at \$0.49 (2s) per 1 000 gal. The total cost of the work will be approximately \$14 100 000 (£3 000 000), but there is little prospect of the system paying more than operating expenses. The annual deficiency to be met by the State will be about \$730 000 (£150 000) with no provision for depreciation. The work has greatly assisted in the development of the district and is, therefore, probably justified by its addition to the revenue and prosperity of the State.

The Kalgoorlie Goldfields Water Supply Project* (Western Australia) comprises a very bold and ambitious plan for supplying with 5 000 000 gal. of water per day the inland goldfields from the coastal ranges across more than 300 miles of arid and semi-arid country.

From the Mundaring Reservoir (capacity, 4 600 000 000 gal., or 18 400 acre-ft.), a series of eight pumping stations, at intervals, force the water through a 30-in. steel pipe line over a distance of 351.5 miles and deliver it at a level 1 290 ft. above the reservoir. The works were completed in 1903 at a cost of \$16 000 000 (£3 250 000).

Since the recent decline of the demand for water for gold-mining purposes, a new and increasing demand has arisen for domestic and stock supplies to the splendid wheat lands adjoining the pipe line the development of which has been held up for lack of water. Not only does this demand absorb all the water thus made available but, in order to provide supplies for further areas, extensions to the system are now being considered.

Artificial Catchments.—The conservation of rain falling on roofs is as old as roofs themselves, but it is believed that the construction of large "roofs" on the ground—designed to catch the full water requirements of large farms—is quite novel.

There are parts of the Mallee country that are quite suitable as a place for people to settle, except that the ordinary channel system of water supply cannot be economically extended to them on account of the excessive cost of pumping, or of lining the channels. This is due to the porous nature of the soil, the absence of holding ground for the farmers' storages, or a combination of these factors. An experiment is being made to form a water supply of artificial catchments by laying sheets of thin galvanized iron directly on the ground discharging into water-tight covered storages.†

Taking the square mile, or 640 acres, as the unit for comparison: It may be assumed that a supply of water is needed for 6 persons, 10 horses, 2 cows, and 150 sheep. The daily consumption at maximum periods is established at 5 gal. per person, 25 gal. per horse, 8 gal. per cow, and 1½ gal. per sheep. Their requirements for a period of twelve months may be set down as 90 000

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXII and CCV.

† The method known locally as the "Ironclad Catchment" has been developed by Messrs. A. S. Kenyon and R. F. McNab, Engineers of the Victorian State Rivers and Water Supply Commission.

gal. net, or a little more than 500 cu. yd., the consumption being distributed throughout the year as shown in Table 3.

TABLE 3.—ESTIMATED WATER REQUIREMENTS FOR 6 PERSONS, 10 HORSES, 2 COWS, AND 150 SHEEP.

Month.	Gallons (Imperial).	Gallons per day.	Rainfall required on 26 000 sq. ft.
January	15 000	500	111
February	13 000	460	96
March	10 000	330	74
April	9 000	300	67
May	6 000	200	45
June	3 000	100	23
July	3 000	100	23
August	2 500	80	19
September	3 500	120	26
October	4 000	130	30
November	9 000	300	67
December	12 000	400	89
Total for year	90 000	250 (daily average)	670

Examination of the available rainfall returns which cover a period of 50 years shows that 26 000 sq. ft. of a perfect catchment will provide the required volume in the worst period on record during that time, and naturally will yield considerably more in the great majority of years. Since the rainfall is not well distributed during the critical periods, a storage capacity of almost two-thirds of the minimum annual requirements is needed.

Experience has indicated that galvanized sheet iron has a considerable life in contact with the soil of the Mallee. Of the 26 000 sq. ft. of catchment, it is anticipated that the roof of the house and sheds will provide about 2 500 sq. ft. The storage tank itself, the volume of which has been fixed at 65 000 gal. to provide a safety margin, will have a roof area of 2 100 sq. ft., leaving 21 400 sq. ft. for the ironclad ground catchment. The tank is lined with cement concrete, 3 in. in thickness, coated with bitumen. An overflow channel leads surplus water away to prevent erosion. The tank is roofed with galvanized corrugated iron, 26 gauge, closely sealed all around and supported on 4 by 1½-in. steel joists spanning the tank. An ordinary hand pump is fitted to the tank, which, together with the catchment, is fenced in with a stock-proof fence. If separate storage, about 7 000 gal., is provided at the house and sheds, the catchment tank may be reduced to 58 500 gal.

To the present, only experimental public catchments have been constructed. For actual farm allotments it is proposed that the Commission supply the iron, cement, timber, and pump; pay the rail freight; land the bending and punching machines, and line and roof the storage tank. The settler is to cart all material to the site, prepare the catchment surface to receive the iron; bend and punch the iron; excavate for the storage tank, about 400 cu. yd.; cart the gravel from railway (24 cu. yd.); and fence in the catchment and tank. If it is done in this manner the estimated cost is:

Catchment iron, 8 tons, and freight	$1 178	
Lining concrete tank 3 in. thick and coating twice with Colas, 284 sq. yd.	486	
Corrugated galvanized iron for roof, 1 ton	146	
Roof framing	97	
Fixing roof, pump, etc.	73	
Anchor timber nails, etc.	49	
Contingencies	306	
Total	$2 335	(£480)

This amount will be repaid by the settler in installments, together with his land purchase price. The settler will have the full responsibility for the management and maintenance of the catchment and tank, and no annual charge for that purpose will have to be met. He may, with profit, add both to the catchment and to the storage, as his circumstances permit.

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PAPERS AND DISCUSSIONS

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LETTING CONSTRUCTION WORK BY COMPETITIVE BIDDING

Discussion*

BY MESSRS. J. E. ROOT, AND SPENCER A. SNOOK.

J. E. ROOT,† M. AM. SOC. C. E. (by letter).‡—This paper contains many points of active interest on the letting of contract work, and the relations between engineers, contractors, and surety companies.

Preparation of Contract.—The ideas presented in Section *A*§ cannot be emphasized too strongly. They practically cover the full field of controversy between the contractor and the owner or his representative, the engineer. The contract form should be brief, clear, and applicable to the work at hand; and, as an instrument, it should represent mutual regard for justice and fairness for each party to the contract. In construction, the instrument should be arranged in proper sequence, of general application, and should contain only matter relative to the particular case at hand. In brief, the one principle for the engineer to follow during its preparation, in order to fulfill completely the objective and purpose which the contract is to serve, should be to make it a logical, comprehensive, readable, reasonable, and workable form.

In the construction of a complicated piece of work the question of "changes" do arise. However, the contract form and the detail specifications need not, and should not, emphasize glaringly that the engineer reserves full authority to use the prerogative of continually harassing the contractor by making constant changes and modifications during the period of construction. The engineer's full responsibility exists during the time in which he is preparing the contract form, specifications, and plans. If, during this interim, he will give the same thought, care, and attention to the contract form, specifications, and plans, which he gives and exercises, in too many examples,

* Discussion on the paper by Edward W. Bush, M. Am. Soc. C. E., continued from February, 1929, *Proceedings*.

† Special Engr., The Cincinnati Union Terminal Co., Cincinnati, Ohio.

‡ Received by the Secretary, February 4, 1929.

§ *Proceedings*, Am Soc. C. E., November, 1928, Papers and Discussions, p. 2437.

after the award of the contract, a better workable understanding and a more harmonious working arrangement will exist between engineer and contractor. If the contractor is ordered by an engineer, or one of his subordinates, to make changes from day to day as the result of plans inadequately prepared, or to modify specifications not prepared to fit the work at hand, how is it possible for a contractor to organize his forces, follow a construction program, use his initiative and ingenuity in the execution of the work, or obtain in the end that to which contractors are entitled, a reasonable profit? The time for changes in the contract form, specifications, and plans is prior to advertising for bids, and the principle upon which engineers should work, should be that no changes are to be made after the contract has been let. If they would adhere to this principle it would immediately establish a definite line of demarcation between the engineer's functions and the contractor's functions during the life of the contract.

Small scale drawings bound with the contract document serve no useful purpose. If it is necessary for the engineer to depict the work to himself by large scale drawings and to correlate all features and details of the problem by various suitable scales, it cannot be presumed that small scale drawings will give full information and enlightenment to a contractor or his engineer representative. The legal connection between the drawings and the contract, can be established by direct written reference made in the contract or in the specifications.

The defaults by contractors are usually from one or a combination of the following causes: (1) Lack of experience or financial responsibility; (2) bid prices that do not represent the actual cost, plus a reasonable percentage for its execution; (3) continued interruption and interference on the part of the engineer; and (4) contingencies arising during the work, which are beyond reasonable expectations.

To meet the first and second anticipated possibilities it is encumbent on contractors themselves to use judgment and discretion in bidding on proposed work. Contractors as well as engineers obtain experience gradually by their increased initiative and desire to expand a little at a time from a moderate to an involved class of work. In time, contractors as well as engineers, establish for themselves a classification and should only be encouraged to compete for work of a class in which they have clearly demonstrated that their past ability will fairly represent the probable successful execution of the work at hand. On this basis it is immaterial whether a 10% or a 5% retainer is withheld for the owner's protection against possible default, because, when expressed in a percentage of the contract, it represents in part the item of profit to the contractor. Carefully prepared balanced bids usually provide adequate funds to meet current obligations and a continuation of the necessary banking credit. The third and fourth possibilities speak sufficiently for themselves.

Established custom may easily and conveniently be modified so as to provide protection for the contractor against the possibility of changes in plans made by an unscrupulous engineer. For example, all bidders should be

allowed to retain the plans until ten days after the award and the successful contractor should be permitted to keep, as his personal property, the plans used to prepare his bid. All deposits made for securing the plans could then be returned, or disposed of as the instructions to bidders may provide.

Usually, the time between the advertisement for bids and the date upon which bids are to be submitted is too short to give a reliable contractor sufficient time for a just and careful consideration of the work involved. After the completion of plans and specifications that have taken months and sometimes years to prepare, engineers are, as a rule, entirely too anxious to receive bids and get a contractor started on the execution of the work. This time period should be reasonable and should vary with the class of work on which bids are to be obtained.

*Section B.**—Under the caption, "Advertisement for Bidders", there is one humiliating feature that the author has failed to mention. Too frequently the notice for a letting is advertised in an engineering magazine of general circulation and it contains each and all of the points mentioned as being of interest to contractors. The result is that "out-of-town" contractors respond. They spend their time and money obtaining information and then submit a proposal which is sometimes the lowest bid. It is not a mere circumstance, nor is it evidence of fair dealing, when the low bidder is informed that "the work has been given to a local contractor". Under those circumstances, the engineer should withhold the advertisement from engineering magazines, and should state in the "Instructions to Bidders" that the contract will only be awarded to a local contractor.

Information for Bidders.†—To obtain fair and equal competition between contractors, full and equal information should be given by the engineer to each and every interested contractor. Too many engineers have favorites. As a rule contractors are less "hidebound" than engineers, and they respond very quickly to fair and honest dealing.

On public work it is usually one of the regular requirements for the engineer to prepare an estimate in detail, and for private work it certainly has been prepared for the benefit and information of the owner. A responsible contractor obtaining the details contained in this estimate will not be governed or controlled thereby in his judgment as to bid prices. If the engineer is fairly competent to prepare the plans and specifications, he is the one most competent to portray the work involved on the plans by his estimate as to quantities, and this is the part of the detailed estimate in which responsible contractors are most interested.

A reasonable period of time between the date of the award of the contract and the date when actual work must be started by the successful contractor should be given. Involved estimates which have taken the engineer a long time to prepare generally also require at least a reasonable period of time on the part of the contractor before he can complete his negotiations with material and equipment dealers, sub-contractors, etc.

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2446.
† *Loc. cit.*, p. 2447.

*Awarding the Contract.**—Bids for public as well as private work should be opened publicly and read. Secrecy plays no part in fair dealing or in the creation of an agreeable, straightforward, working relationship between engineer and contractor.

Certainly, in fairness to those who may be equally qualified by experience and financial responsibility, the contract should be awarded to the lowest bidder. Furthermore, when alternate items are included on the plans and in the proposal, the work should be awarded on the basis of the lowest alternate. When alternates are specified by the engineer, it is to be presumed, and should be only so considered, that each and all of them are equally adaptable for the work to be constructed. If this is not the case the engineer should declare himself clearly so as to avoid misleading the contractor and exciting the hostility of those directly interested in the alternate.

SPENCER A. SNOOK,† ASSOC. M. AM. SOC. C. E. (by letter).‡—This paper is timely and brings attention to carelessly worded specifications and to plans insufficiently detailed and designed. It also directs thought to the relation of surety bonds to contracts. It is desirable to have the honest co-operation of all parties and co-ordination of all factors involved.

The writer believes that in the majority of cases, the unit price bid is the best for all concerned. Changes which involve more than a simple reasonable increase or decrease of any item bid are sometimes necessary in the contract. Such changes are, or should be, beyond the control of the owner's engineer because of conditions not possible to foresee. Changes other than these should not be necessary, because plenty of time should be given to study the design, to prepare the drawings in sufficient detail and to obtain the best borings and other sub-surface information. The engineer does not always have sufficient time to make complete plans; he is often forced to get them done within a certain time limit. Some engineers do not think it necessary to go into details, because they know that those who prepare shop drawings like plenty of leeway. This is a viewpoint that is favored altogether too much. With detailed plans available, the checking of shop drawings is made easier and quicker and the engineer is more likely to get what he really wants. Furthermore, engineers who know how to make plans with enough details will not do that work for the money customarily offered. It is the same old story—"if you want something good you must pay for it".

The specifications cannot be written too clearly and concisely. Provision should be made for supplementary written agreements between the contracting parties for necessary and unusual changes. A description of the work to be done and possible obstacles to be overcome should be stated plainly.

To demand good workmanship alone might lead to considerable argument. If good results are specified, only good workmanship will produce them, generally speaking. However, when certain methods produce the good results desired, it seems best to call for those methods to be used, because the results otherwise may be considered good, but not just what was wanted. Also, it is

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2452.
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‡ Received by the Secretary, February 23, 1928.

important, very often, to call for materials of a certain kind, especially for concrete of a certain strength, or structural steel, bearings, etc., of a certain make, since good results cannot be judged by visual inspection.

The quantities bid upon should be calculated accurately to the best of the engineers's ability and information, since he knows the construction involved. The bidder should be furnished with all known information and requirements, and anything that is not positively known to be accurate should be so marked.

It would seem fair to ask the contractor to guarantee the work done under his control by means of a surety bond. There are some owners who may not know when a bid is too low or that the contractor is not a responsible one. Such people need some form of protection and for them a surety bond is desirable. The surety bond should be regarded as a guaranty against claims and damages and for the prompt execution of the work with the results specified, as all are within the contractor's control. If the owner never had any worries about these things, he would not require a contractor to obtain a surety bond. There must be some risk whenever a surety bond exists with a contract. The amount of that risk to be taken, respecting the responsibility of the contractor to pay for his labor and materials and to finish the work as desired, is something for the surety company to decide when writing its policy. If it takes good risks, the premiums should be low, but if it just wants to sell policies without knowing how responsible the contractor is, the premiums will be necessarily high.

Cash, or the bidder's check for bidding, is preferable to bid bonds. The acceptance of letters which guarantee a bond to the successful bidder is an undesirable practice and should be eliminated because of the possibility of collusion.

It might be well to keep the surety company advised on monthly estimates and changes in contract, having in view the ratio of work completed to that to be done within the given time limit.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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SPECIFICATIONS FOR TRANSIT TRAVERSING AND STADIA LEVELING

Discussion*

BY MESSRS. C. V. HODGSON, C. KELSEY MATHEWS, and HOWARD S. RAPPLEYE.

C. V. HODGSON,† M. AM. SOC. C. E. (by letter).‡—The author has outlined a logical method of determining the specifications for accuracy in a survey and goes further by presenting the mechanics of designing them. The method shown is not only logical, but it accords well with engineering principles. In designing a bridge the engineer must know the purpose to be served, the load to be provided for, and then must decide what type of construction will best meet those requirements. The detailed designing can then be begun and the strength of the individual members computed. Similarly, in a survey, the purpose must first be clearly understood, the requirements of accuracy and specialized data to be met must be formulated, and then decision as to the type of survey and kind of instruments can be made. The specifications for the strength, or accuracy, of the individual operations must then be determined.

Although the method of writing specifications for a survey described by the author is not new in principle, it shows in a clear manner the method of combining the errors attributable to each of a number of associated factors in a surveying operation and emphasizes the important distinction between a systematic and an accidental error. This distinction is the most essential factor to be borne in mind by the surveyor, except for the necessity of adopting methods which will avoid the possibility of mistakes or blunders remaining undetected. One wishes that the sentence in Professor Rayner's definition§ of "Mistakes" were always true—that "they are detected by checking the work and are eliminated by applying corrections." Despite elaborate systems of

* This discussion (of the paper by W. H. Rayner, Assoc. M. Am. Soc. C. E., published in December, 1928, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, January 28, 1929.

§ *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2640.

checking, the chances for mistakes have been found by all surveyors to be surprisingly numerous, and there is usually the possibility that some blunder will not be detected by the field checks or by the office computation.

The examples given in the paper* show strikingly the difference in the rates at which systematic errors and accidental errors accumulate. A systematic error of ±0.1 ft. per tape length with a 100-ft. tape, will give an error of 1 part in 1 000, and this error is independent of the number of tape lengths. An accidental error of +0.1 ft. per tape length, in 100 tape lengths, will represent a probable error of ±1.0 ft., or 1 part in 10 000. The principal aim of the surveyor in securing a specified accuracy is to reduce systematic errors to a permissible limit by his methods of measurement, and to make certain that what he regards as accidental errors do not tend to become systematic ones. For instance, in Table 4† the estimated tension is taken to vary from the standard tension by a probable error not greater than 3 lb. and this is considered to be an accidental error. However, if the estimation of tension should be consistently higher or lower than the standard tension, as would conceivably be the case in actual practice, a systematic error would be introduced which would accumulate at a different rate than an accidental error. The same principle would apply to the estimation of temperatures or slope of ground.

It is probable that some may question the adequacy of the definition given for "Error",‡ which states that "an error always is a quantity of which both the magnitude and the sign are unknown." That part of the definition relating to the sign does not seem to be correct or essential, because the error due to the wind effect of a suspended tape is always such as to make the measured length too large, as is also the error due to lack of alignment of a tape. Other examples of the same tenor could be cited. Also, many will prefer a definition for "Systematic Error" which will provide that it need not be always of the same (unknown) sign, but instead that it is defined as following some definite law or relation. For instance, the temperature error in a series of tape measurements may conceivably cause both minus and plus errors at different parts of the same series of measurements, because the temperature varies above and below that at which the tape is standard, but since the error varies in accordance with some definite law, it may be called systematic.

C. KELSEY MATHEWS,§ ASSOC. M. AM. SOC. C. E. (by letter).‖—The author has presented this topic in a manner that should appeal to the engineering mind, inasmuch as the subject of specification writing for the assurance of proper results by an economic process constitutes a major part of the design of all engineering projects. Engineers have been more or less altruistic in attempting to obtain the ultimate economy for the client through refinements in design and construction and have neglected to apply the same principles of economy in their own work. The principles set forth in the paper may be applied in a practical way to reduce expenses.

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2640.
† *Loc. cit.*, p. 2651.
‡ *Loc. cit.*, p. 2640.
§ Assoc. Engr., Burns & McDonnell Eng. Co., Kansas City, Mo.
‖ Received by the Secretary, February 14, 1929.

A large part of the expense of an engineering business may be for field surveys. Generally, the conception of the project originates with some one man in the organization, and through him is determined what field information is to be obtained. The manner in which this is to be achieved is generally left to an assistant whose only qualification for directing surveys may be several years' experience in manipulating surveying instruments.

The lack of knowledge of the purpose of the various parts of the survey, the various degrees of precision required and the methods necessary to secure it, generally results in financial loss to the business. The principal in charge of a project should have a thorough knowledge of these factors, and detailed "specifications", either written or oral, depending on the scope of the project, should be given the assistant in charge of surveys, so that the required precision may be obtained at the least cost. In this manner only the manipulation of instruments is left to the field men.

The errors in such manipulation are those of observation and judgment; they are "accidental" in character, and the effect on the result is comparatively small if the surveyors are skilled and experienced. The important factors will be disposed of as "systematic errors and cumulates" by the observance of the requirements of the "specifications". Provision should be made in the specifications for proper methods of checking against mistakes.

There are two obstacles to be overcome in profiting by specification writing for surveys. The first and most important is the ignorance of the "principal" as to the existence of the various sources of error. The second is the reticence of self-styled surveyors to observe new ideas which are not consistent with habits acquired through years of manipulating instruments.

The author's analysis and definitions should provide a liberal education, tending to overcome the first obstacle, if the "principal" can be convinced of the economy which will result. The detailed analysis set forth will actually be a post-graduate course to many engineers after learning that a transit may have a "stadia factor interval" different from 100, and that such difference is of much more importance than exceeding care in reading the intercept. In one out of twenty transits of various makes and ages, the writer found that ten had a factor of 100; seven, of 99 or 101; two, of 102; and one, of 105.

The writer knows of engineers who have insisted that plans and specifications shall obtain consistent economy for the client; who, nevertheless, have taken no cognizance of the possibly large cumulative error involved in the stadia interval. At the same time, they read horizontal angles in the same traverse with a magnifying glass on a 1-min. transit.

One topographical survey for a large hydro-electric project, which kept four parties in the field for six months, was made with no other important instructions than to "get two miles of traverse per day". Several days were lost in one instance in computing and re-surveying one traverse, run with two transits, which failed to close "satisfactorily", only to find later that one of the transits was "out of adjustment"; that is, the interval factor was not 100 as assumed. Numerous examples of such uneconomical procedure may be cited, all of which might be eliminated by "specifications".

The application of the laws of accidental errors to surveying measurements and to the propagation of errors through a combination of different measurements and calculations is certainly more reasonable and consistent than the application to some other engineering data, such as flood flows. This is especially true considering the limited records available for most engineering calculations.

The probability curve is determined theoretically only by an infinite number of observations. In determining the probable frequency of a certain maximum flood, based on a limited number of observations which are under no definite control, the truth is only confused by a mass of probability figures. In shooting at a mark, or in estimating the slopes of the various courses of a traverse, the observations are under control, and the deviations from true value will be truly accidental in character. The laws of probability, therefore, will apply with greater mathematical certainty even if the observations are few in number.

The value of specification writing for transit traversing, stadia leveling, or any other form of surveying, can be realized only if the field men understand and comply with the specifications. The wording and the minuteness of detail necessary will depend on the type of men for whom they are written.

The writer believes that the "specifications" as outlined by the author in the examples given should be classed as calculations and notes made by and for the principal in charge as a guide for writing the actual specifications. These calculations and notes might be given the assistant in charge of field work so that the spirit and purpose of the actual specifications might be more easily understood. The knowledge that partial derivatives and a few square roots had been introduced might be confusing to some "practical" surveyors and might not secure the co-operation desired.

Consider, for instance, the data given in Example 6.* The eight specifications listed under Section (*d*) are the principal's notes. The suggested working specifications to be handed to the field assistant would read as follows:

(1) Compare the tape with standard by stretching between hubs (set permanently near office) at a pull of 12 lb. determined by a small spring balance. Note correction in length to nearest 0.01 ft. and note air temperature from the thermometer to nearest degree, before leaving office.

(2) In the field, estimate temperature of tape at beginning and completion of chaining and note to nearest 10 degrees.

(3) Estimate average slope by aid of hand level for each critical change in the slope and note to nearest 1 per cent.

(4) and (5) Hold chain parallel to the general slope of the course, or on the ground, maintaining as nearly as possible, by estimation, a 12-lb. pull.

(6) Set flag on forward station and have rear chainman align head chainman therewith when deviation is apparently more than 1 ft.

(7) Stick pins or mark, without elaborate care, so that the station is marked within 0.05 ft. of apparent exact position.

(8) Use 1-min. transit; measure all angles to right with telescope direct; read and record angle to nearest minute without magnifying glass; reverse

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2650.

telescope, reset plates, and read and record angle to right again; read needle and record to nearest degree.

Checks.—(1) In taping, both chainmen shall calculate all stations from pin count and call and check; (2) both chainmen shall call and check all pluses; (3) in measuring angles, duplicate all readings; and (4) read the compass.

Specifications for calculating and removing cumulates should be written, or, preferably a blank form should be provided for entering the corrections with the proper sign, and the corrected chaining.

The sixth specification under Section (*d*)* stipulates that, "errors due to variations from true alignment (E_0) may be neglected". The writer has included an item (corresponding to Example 6) that controls the precision desirable for this source. The author has stated in Item (*d*), under "Sources of Error in Chaining",† that, "the effects of errors of alignment are exactly similar to those of grade, except that they can be minimized much more easily." The writer recognizes the error due to alignment in the sense of chaining along a curved line between two traverse hubs rather than along a straight line, and acknowledges that this error may be neglected if a sincere attempt is made to chain toward the forward hub. The source of error which should be controlled is the "zigzag" deviation of the ends of the tape from the general line of chaining independent of its general curvature. The first error is similar to that introduced by chaining parallel to, or on, the general ground slope without correcting to horizontal. This error would amount approximately to 0.1 ft. in Example 6 if the average offset from a straight line between hubs were 3 ft. The second error is similar to that introduced by the ends of the tape not being at the same elevation in attempting to measure horizontal distance directly, or by not being in a plane parallel to the general slope in chaining according to the problem assumptions. This error would amount approximately to 0.25 ft. in Example 6 if the average "zigzag" deviation of the end of tape from the general line of chaining were 1 ft.

An interesting point is developed in connection with this subject when a comparison of the precision obtainable by chaining on the ground and removing the slope cumulate is made with that obtainable by attempting to hold the tape horizontally and set the point by plumb-bob.

If the grade is estimated for every material change in the general slope along a traverse with an average error of 1% (which may be done quickly with the aid of a hand-level), the error in the corrected length of traverse will be "accidental" and will amount to the square root of the number of estimates times the average error in length per estimate. In Example 6, if there are 25 estimations of slope (1 per station), the probable error due to estimating grade would be $0.06 \text{ ft.} \times \sqrt{0.25} = \pm 0.30 \text{ ft.}$

The author has assumed that, for the remaining half of the traverse, the tape is held unsupported and horizontally, but has allowed nothing for the error introduced because of the ends of the tape not being in the same horizontal

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2651.
† *Loc. cit.*, p. 2643.

plane. This error would be 33 × 0.005 ft. = 0.17 ft., if the deviation from horizontal were 1 per cent. If the average slope were 3%, or less, the precision obtainable by the two methods would be nearly the same, with the time advantage in favor of chaining on the ground or parallel to the general slope.

The determination of the stadia constant within a probable error of ± 0.1 is rather unusual, and the method of obtaining it should be outlined in detail for field specifications. These might read as follows, for Example 7:*

"Ten arbitrary points on level ground and in line from the transit shall be marked up to approximately 1 000 ft. The intercept shall be read to the nearest 0.01 ft. on each distance. The distance to each shall then be taped and recorded to the nearest foot. The value of K for each distance shall then be calculated from the formula, $K = \left(\frac{D}{S} - 1\right)$, and recorded to the nearest 0.1. The mean of the ten values shall be calculated to the nearest four significant figures and used for K."

These suggested specifications for determining K are calculated from an assumed probable error of ± 0.003 ft. in reading the intercept and of ± 0.3 ft. in distance, which is equivalent to a probable error of ± 0.3 ft. in K. The probable error in the mean of ten determinations would then be approximately ± 0.1 ft. The probable error in K could be checked more accurately by computing the residuals for each determination afterward.

It is necessary to have the proper perspective with regard to the surveys for a project, in the same sense as a proper perspective is necessary in the conception of the project as a whole. In other words, an economical survey cannot be made by an unrelated system of measurements any more than a structure can be built economically by adding arbitrary units to a nucleus. The designer must visualize the component parts assembled, and must know their correlation.

The author is to be congratulated on presenting a paper containing the fundamental principles necessary to attain the proper perspective with regard to the surveys, and also practical suggestions for their direction and execution.

HOWARD S. RAPPLEYE,† ASSOC. M. AM. SOC. C. E. (by letter).‡—This paper deals with a phase of surveying that has received little attention in the past. With the possible exception of organizations charged with the execution of extensive surveys over large areas, engineers have fallen into the habit of using more or less rule-of-thumb methods of deciding what instrumental equipment and field procedure is to be used for a given survey. It is undoubtedly true that a vast amount of time and effort can be saved by properly drawn specifications regarding the necessary accuracy of the various steps in the making of a survey. It stands to reason that such specifications should be based on thorough studies of the effect of the various sources of error, considered singly and in combination, on the accuracy of the final results.

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2652.

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‡ Received by the Secretary, February 15, 1929.

The writer believes, however, that specifications should be drawn for the several classes of surveys rather than for individual surveys. Except, perhaps, in the case of property surveys, it is not usually known beforehand what the shape of the traverse will be, nor is it usual to have any very detailed knowledge of conditions to be encountered along the route of a given traverse. This is especially true in certain parts of the country where, within the small space occupied by a farm of 100 acres, the character of the land may vary from almost perfectly flat bottom-land to the most rugged side-hill topography.

The concept that an error must be an unknown quantity in both sign and magnitude* is open to discussion. The error due to phase in pointing on a signal is certainly systematic, but its sign is not necessarily unknown. The definition* of a "systematic error" is likewise open to question, although it is true that it derives its name and character from the fact that its algebraic sign remains the same. The fact that this sign is always in doubt seems to be subject to debate. Take, for example, a cross-wind blowing against an unsupported tape. This certainly introduces a systematic error in the measurements, but the sign of the error is positively known since it always shortens the distance between the terminal marks of the tape and, thereby, causes lengths measured under these conditions to be too long.

That the effect of a systematic error cannot be eliminated by systematic field methods appears to be also a debatable subject. Suppose that a line is taped along a smooth concrete roadway, and a finely pointed pencil is used to mark the tape ends. It is conceivable that a tapeman might have a persistent tendency to mark to the right of the graduation at the end of the tape by a small amount. This tendency certainly would produce a systematic error. Now, suppose the measurement to be progressing toward the forward contact man's right, as he kneels by the tape. If he has a tendency to mark to the right he will be marking ahead, and his resulting measure will be too short. Suppose, then, that the measure is repeated in the backward direction, but that the observer stays on the same side of the tape. His tendency to mark to the right will then cause him to mark behind and his measure on the backward running will be too long. The mean of the two measures should then be free from his personal equation or systematic error in marking.

The first part of the definition* of a "cumulate" represents what the writer is in the habit of referring to as a correction. Any systematic error in surveying that can be eliminated by calculation takes on the character of a correction and does not affect the agreement of the final results. Any source of systematic error which can be, and is, eliminated by systematic field methods never appears in the final discrepancies at all. It would seem to be wise to retain the term, "correction", as referring to a systematic error for which an allowance is made by computation, and, since the other feature of the definition of a "cumulate" fails to enter the final results when proper field methods are used, it would seem that the necessity for the new term is slight at best.

"Mistakes"* are the result, not only of confusion in the mind of the observer, but also of confusion in the mind of the recorder or from misunderstandings

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2640.

arising between them. If mistakes are of a size considerably larger than the errors of observation, they are detected because of their lack of conformity when measures are repeated but, in the case of small mistakes of a size equal to or only slightly larger than the errors of observation, they may be easily mixed up with legitimate errors and be treated accordingly.

Regarding temperature errors in tape measurements: If the temperature is assumed to remain constant, as suggested,* the error is systematic to be sure, but if the assumed temperature is between the maximum and minimum actually encountered during the course of the work the sign of the error will change, depending on whether this temperature is above or below the actual temperature at the time, and the resulting errors will not be strictly systematic throughout the whole traverse. It is noted that, in Table 1,† Professor Rayner has computed values of E_t to the nearest 0.005 ft. per 100 ft. The writer suggests that this should be the nearest 0.001 ft., in order to be consistent with values of E_s in the same table.

To use the telescope reversed on the backsight and direct on the foresight at the odd numbered set-ups and direct on the backsight and reversed on the foresight at the even numbered set-ups will merely serve to place the error of adjustment of the line of collimation alternately plus and minus as the instrument progresses around the traverse.‡ It seems to the writer that it would be better to let the error accumulate in a closed traverse and then eliminate it by distribution after the traverse is closed. Theoretically, this would give the correct angle at each station where the suggested procedure would make each angle in error by the same amount, but with the sign changing at each set-up along the traverse. In work in which considerable accuracy in angles is required, a reversal at each station is desirable; reading double deflections or using the transit as a direction instrument will accomplish the desired end.

In the comparison of the computed probable errors, under the heading "Errors in Chaining and in Angles Combined", the author states§ that the systematic errors were eliminated from the closed traverse, but could not be eliminated from the open traverse. This carries the idea, by inference at least, that systematic errors in an open traverse are more dangerous than in a closed one. It is true that the systematic errors which operate throughout the entire closed traverse do not appear in the final closure, but they are present, nevertheless. If a systematic error of 1 part in 1 000 is present in a closed traverse the accuracy of the survey is only 1 part in 1 000, even if luck is with the surveyor and he happens to get a perfect closure on his return to the initial station.

Under Example 6, Sub-division (*a*),‖ the permissible error (1 ft.) is substituted as the probable error in a formula. To the writer this seems to be the misuse of a perfectly good formula. The total permissible error should not be assumed to be equal to the probable error.

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2642.
† *Loc. cit.*, p. 2644.
‡ *Loc. cit.*, p. 2645.
§ *Loc. cit.*, p. 2649.
‖ *Loc. cit.*, p. 2650.

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PAPERS AND DISCUSSIONS

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COMMENTS ON THE DESIGN OF SLUDGE DIGESTION TANKS

Discussion*

BY MESSRS. HARRISON P. EDDY, JOHN R. DOWNES, WILLEM RUDOLFS, EDMUND B. BESSELIEVRE, and JERRY DONOHUE.

HARRISON P. EDDY,† M. AM. SOC. C. E.—The changes which have been taking place in the art of sewage disposal since about 1918, are somewhat radical. For many years, the primary effort was to produce an effluent which would satisfy local conditions, and the problem of sludge disposal, while recognized as being troublesome, was subordinated in a measure. The present tendency is clearly toward more complicated treatment plants, and the division of the processes into a number of stages.

Formerly, little attention was paid to the matter of odors produced at treatment plants. It was common practice to discharge raw sludge into lagoons or on to open drying beds, often resulting in the dissemination of offensive odors. At present, determined effort is made to prevent the escape of odors in so far as possible. Sludge digestion is now commonly practiced to accomplish this.

It has long been recognized that temperature has a material effect on decomposition, but sufficient attention has not been given to this phase of the digestion process. It is well known that, in the colder climates, streams which are so polluted as to create offensive odors in summer are relatively inoffensive in winter.

It has not always been recognized that the storage of sewage solids on the stream bed in winter is accumulating trouble for the following summer. In the colder climates of this country such solids are subjected to a refrigerating process for perhaps six months of the year. The temperature of the river water is so low that it prevents the solids from changing materially, but when summer comes, with an increase of 65° or more, in water temperature, decom-

* This discussion (of the paper by Richard H. Gould, M. Am. Soc. C. E., presented at the meeting of the Sanitary Engineering Division, Asheville, N. C., April 21, 1927, and published in December, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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position of the accumulated solids creates a load on the stream which is greater than the current load.

The digestion of sludge in unheated tanks takes place under similar conditions, but by controlling the temperature, as Mr. Gould has indicated,* active digestion can be maintained continuously and it is possible materially to reduce the storage capacity required. Heat may be provided by burning the gas evolved from the digesting sludge; but this introduces additional complications because of variations in the rate of gas production, the need for regulation of pressure, and the necessity for adequate precautions against accidental explosion.

Atmospheric oxygen, mingling with the gas in a tank, may form an explosive mixture. Such mixtures, containing between 5.5 and 12.8% methane, are explosive, and may be produced by many conditions such as those following the partial removal of sludge from a tank.

There is difference of opinion as to the mechanical equipment which should be provided for separate sludge-digestion tanks. It is practically impossible to introduce the sludge uniformly throughout the tank. This difficulty may be overcome in a measure by artificial mixing. A mechanical stirring device may be placed at the bottom of the tank where solids are supposed to accumulate, but the solids may accumulate in the upper part of the tank, in which case stirring equipment would be more effective at the top. Experience has also shown that there may be an accumulation of solids in the middle of the tank, with comparatively clear water above and below. These conditions are probably due in large measure to the kind of biological action taking place, although temperature and other physical conditions undoubtedly are important factors.

Some biological processes are carried out most advantageously at certain rather definite points of acidity or alkalinity. It is logical that there should be an optimum reaction for the digestion of sewage sludge. The reaction may be regulated by the addition of chemicals. Having established favorable digestion, it is important to keep a quantity of sludge in the tank sufficient for seeding the freshly deposited sludge.

In developing the art of sewage treatment, in an effort to prevent offensive odors, to secure a high degree of purification, and to handle the sludge under advantageous conditions, engineers are introducing processes and equipment which add operating complications. These also add to the cost of design, construction, and operation.

It is difficult to secure funds for proper operation, but it is more difficult to secure operators who have the technical knowledge and skill required to operate such plants successfully. This country has advanced from the simple, "fool-proof" sewage treatment plant of the Nineties, of which very little was required except good luck, to the highly complicated mechanical plants, like that at Milwaukee, Wis., and like some of the separate sludge digestion plants. The difficulties ahead in the field of operating these plants constitute one of the most serious problems confronting the engineer practicing in this field.

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2660.

JOHN R. DOWNES,* ESQ. (by letter).†—In the writer's opinion, the work of Mr. Gould stands out as of the greatest importance to the art of sewage disposal in that it sums up so accurately the present-day knowledge and information on sludge digestion.

Those who have been intimately connected with scientific investigations have realized for some time the necessity of having the *status quo* of developments presented from the engineering standpoint. Mr. Gould has presented the matter so ably that any analytical discussion would simply tend to befog that which he has made extremely clear.

He suggests that inert material might settle out in digestion tanks to the extent of decreasing the available digestion space. In reply it may be stated that in tanks of the design shown by Mr. Gould‡ which are practically flat-bottom tanks, only from 2 to 4 in. of gritty, inert material has settled out during a period of two years, notwithstanding the fact that a grit chamber installed at the end of the two-year period removed about 6 cu. ft. of grit daily from the incoming sewage. In the settling tanks and flow channels the grit tends to separate from the sewage and from the thin watery green sludge, but, in the digestion tank, it becomes rather intimately incorporated in the more viscous, ripe sludge and is carried out with the latter. This specific information was not available at the time the paper was prepared.

WILLEM RUDOLFS,§ ESQ. (by letter).‖—This admirable paper is one of the best attempts to bring together and apply the findings in this field during the last few years. In this case a discussion from the standpoint of an investigator must confine itself to supporting evidence and possibly to the emphasis of some minor details of practical interest to the designing engineer.

The course of digestion of fresh sewage solids is essentially the same in septic tanks, two-story tanks, and separate sludge digestion tanks. However, the difference in the construction of these tanks indicating their use is of great importance in connection with the digestion capacities and the manner of operation. In septic tanks and two-story or Imhoff tanks, two distinctly separate processes are combined: (1) Sedimentation, which is a purely mechanical process; and (2) digestion, which is a purely biological process. Although there is no relation between the two processes, in a septic tank both must take place within the same tank, and in the case of an Imhoff tank there is a more or less intimate connection between the upper and lower compartments. In an Imhoff tank, sewage solids settle and fall through the slots and are supposed to remain there until they are decomposed. This is only partly true, since the solid particles (except at the bottom of the tank when very old sludge is present) are in constant motion, either up or down, depending on whether or not the particles carry entrained gas. Part of this material as well as a part of the liquefied material is displaced by the incoming fresh material. The displaced materials leave the tank with the effluent. From the

* Supervising Engr., The Plainfield and Dunellen, N. J., Sewage Disposal Works, Bound Brook, N. J.

† Received by the Secretary, December 29, 1928.

‡ *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2663, Fig. 4.

§ Chf., New Jersey Sewage Experiment Station, New Brunswick, N. J.

‖ Received by the Secretary, January 14, 1929.

standpoint of best possible results the processes of sedimentation and digestion, unlike in principle, cannot be combined because, in order to obtain the best possible sedimentation, the suspended solids and partly liquefied material should be prevented from rising and contaminating the effluent.

With separate sludge digestion tanks the replacement discharge does not take place; or, in other words, practically the whole volume of material added must be withdrawn. Studies on operating tanks that treat fresh solids have shown that only from 4 to 8% of the volume is reduced, due to biological activities and about the same percentage when activated sludge is digested. Obviously, when thin fresh solids are kept in a tank without further concentration the digestion capacity would be much greater than in a two-story tank. Fortunately, fresh solids that are added to a tank separate quickly from the water. This takes place in separate sludge digestion tanks as well as in Imhoff tanks, and thickened sludge in separate sludge tanks can be obtained by decantation, whereas in Imhoff tanks the decantation process takes place automatically, although it cannot be controlled. With control, the operator is able to prevent contamination of the effluent from the settling units and thereby prevent possible nuisances due to odors and difficulties in overloading of sprinkling filters.

Little information is available on the condensation of sludge from domestic sewage. Decantation of the comparatively small quantities of supernatent liquid is rather easy, but the question arises at once as to how far such displacement of liquor may be carried. Exhaustive laboratory experiments have shown that sludge concentration can be practiced to a high degree without detrimental effect to digestion activities.

The longer the digestion schedule, the larger is the digestion capacity required. Assuming good operation, the effective digestion capacity of a separate sludge digestion tank can always be the same or less than for an Imhoff tank. As soon as control is practiced the digestion capacity of a separate sludge tank is considerably less. This is especially true in cases in which heat is applied. Heating of a separate sludge tank is a comparatively simple matter, whereas experiments conducted by Dr. Imhoff to heat the digestion compartment of an Imhoff tank failed because the temperature could be raised only from 2 to 3° Fahr. without excessive heat supply. Moreover, too great quantities of hot water introduced would tend to produce a thin sludge.

In the past, the designer did not have available a number of fundamental facts. Several factors influencing digestion, such as seeding, temperature, and reaction, have been investigated and are now largely available to the engineer. A few curves are submitted herewith to emphasize and illustrate the more important single factors.

The most important single factor known to the writer at present is the inoculation or "seeding" of raw sewage sludge with sludge that has undergone decomposition. Such sludge has a well-balanced group of organisms and their enzymes, responsible for rapid decomposition, contain the necessary chemical constituents to counteract the effects of intermediate decomposition products formed during the early stages of digestion (Fig. 7). Although the total

quantity of gas produced per given quantity of organic matter is not influenced by seeding, the material not inoculated requires about six times longer for digestion than the properly inoculated sludge at the same constant temperature of 70° Fahr.

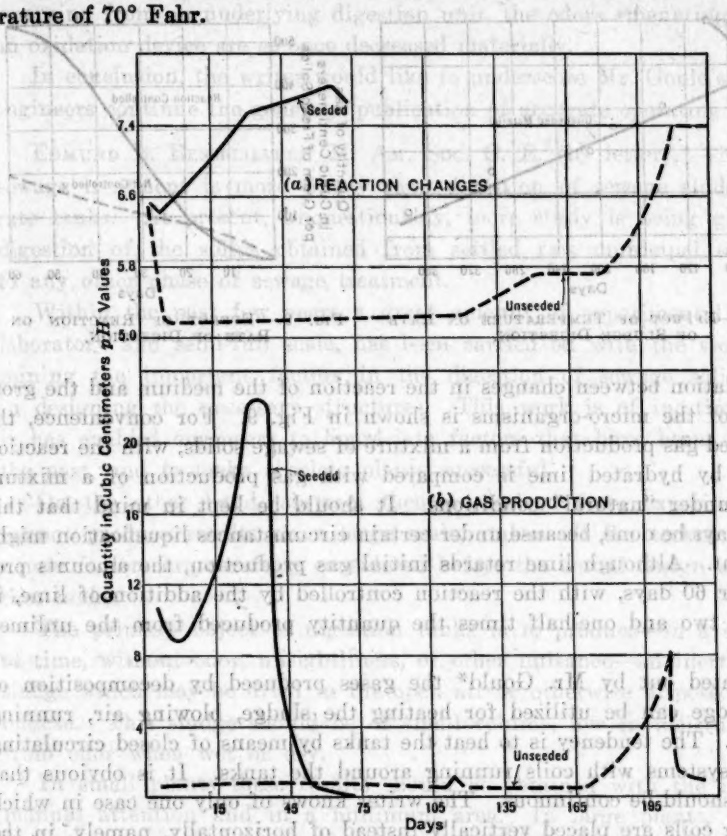

FIG. 7.—EFFECT OF SEEDING ON THE RATE OF SLUDGE DIGESTION.

The effect of temperature is second in importance (Fig. 8). Measured by the digestion time required, unseeded fresh solids at 50° Fahr. require six times longer than material digested at 82° Fahr., which is considered the approximate optimum. Only at very much higher temperatures could it be possible to reduce the digestion time further. With properly seeded mixtures the digestion time at 70° Fahr. is nearly five times shorter than at 50° Fahr. Although the information available is comparatively scarce, the average yearly temperature taken at a number of sewage disposal plants varies between 52 and 58° Fahr. It is evident, therefore, that sludge heating is of great importance in a northern climate, although it should not be forgotten that the average yearly temperature of the material in the sludge digestion tanks at Baltimore, Md., is only 52.5° Fahr. This brings up the important question of proper insulation of digestion tanks. If tanks are built above ground there should be at least an earth fill around them, although it might be better if

some one would devote time to insulate them properly. Insulation is not only important when tanks are built above ground, but also when tank contents are influenced by ground-water.

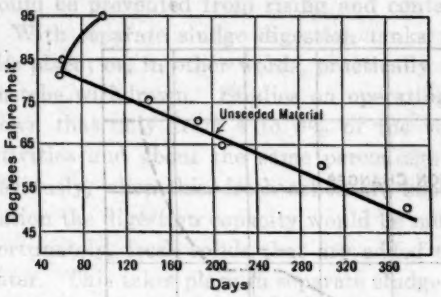

FIG. 8.—EFFECT OF TEMPERATURE ON RATE OF SLUDGE DIGESTION.

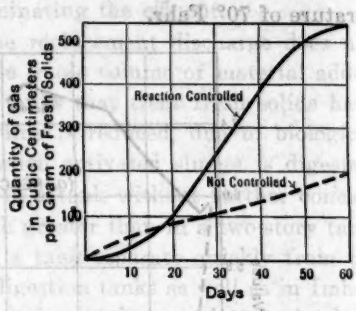

FIG. 9.—EFFECT OF REACTION ON RATE OF DIGESTION.

The relation between changes in the reaction of the medium and the gross activities of the micro-organisms is shown in Fig. 9. For convenience, the accumulated gas production from a mixture of sewage solids, with the reaction controlled by hydrated lime is compared with gas production of a mixture digesting under "natural" conditions. It should be kept in mind that this cannot always be done, because under certain circumstances liquefication might be too great. Although lime retards initial gas production, the amounts produced after 60 days, with the reaction controlled by the addition of lime, is more than two and one-half times the quantity produced from the unlimed material.

As pointed out by Mr. Gould* the gases produced by decomposition of sewage sludge can be utilized for heating the sludge, blowing air, running pumps, etc. The tendency is to heat the tanks by means of closed circulating hot water systems with coils running around the tanks. It is obvious that such coils should be continuous. The writer knows of only one case in which the heating coils are placed vertically instead of horizontally, namely, in the new tanks built for the digestion of screenings at Milwaukee, Wis. Offhand it would seem that vertical coils are better from the standpoint of water circulation and possible caking. Experiments made at the New Jersey Sewage Experiment Station have shown that water heated to 140° Fahr. caused no caking. Further experiments along this line would be of interest.

Since nearly the same volume added to a separate digestion tank must be withdrawn, means for withdrawal of supernatent liquid are important. An overflow at the top might carry with it some suspended matter. Additional "take-offs" at different levels would facilitate operation.

In addition to what Mr. Gould states† about permissible loading of tanks based on sludge digestion capacity per capita, it should be kept in mind that with improved settling of suspended solids the volume of sludge to be handled by the digestion tank increases. Naturally, when more solids are retained in

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2660.
† *Loc. cit.*, p. 2658.

a tank the volume of sewage to be handled by a following oxidation device (like sprinkling filters) can be increased. Moreover, when solids are settled without contamination of the tank effluent by the partly decomposed material emerging from an underlying digestion unit, the odors emanating from such an oxidation device are at once decreased materially.

In conclusion, the writer would like to underscore Mr. Gould's plea,* that engineers continue the generous publication of accurate operating data.

EDMUND B. BESSELIEVRE,† M. AM. SOC. C. E. (by letter).‡—No phase of sewage treatment is more timely than digestion of sewage sludge in separate tanks. At present, unquestionably, more study is being given to the digestion of the solids obtained from settled raw municipal sewage than to any other phase of sewage treatment.

Within the past few years a great amount of experimental work, both laboratory and semi-full scale, has been carried on with the view to determining the important factors in the digestion of sewage solids, for use in designing the necessary structures. This work is of inestimable value; it has enabled engineers to appreciate factors that have been neglected in the past, and to make obsolete plants successful.

On the other hand, abstract theories derived from experimental work, cannot produce results on a large scale, unless all the technical and economic factors are carefully considered before the actual design of the digestion tanks.

The primary object of digestion tanks is to produce—in a short period of time, without odor, unsightliness, or other nuisance—an inert, inoffensive sludge which may be dried in the open air or otherwise disposed of without offense. This sludge should be of small volume, low in moisture, and free from odor when wet or dry.

In small plants these results should be obtained with the minimum of manual attention and in a minimum area. In large plants, area is perhaps the greatest consideration because adding one or two men may be cheaper than greatly increasing the area.

Observation made in the period since 1925 clearly demonstrates that such results may be obtained. When the digestion of the organic solids is complete, the resultant sludge may represent only 40% of the original dry weight. This lessened volume of wet sludge to be discharged on the drying beds, in turn means that a smaller area of beds is required. With the modern plan of covering beds with glass, continuous use of all the drying area may be obtained regardless of weather, further reducing the area required. In small plants, where land is cheap, glass-covered beds at about $3 per sq. ft. may not be more economical than open beds, but this would seldom hold true in a larger plant.

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* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2665.

† San. Engr. and Mgr., Eastern Territory, San. Eng. Div., The Dorr Co., Inc., New York, N. Y.

‡ Received by the Secretary, January 5, 1929.

The main considerations to be given careful thought in designing sludge digestion tanks are:

1.—Volume.
2.—Shape and number.
3.—Simplicity of construction.
4.—Operating devices for:
(*a*) Controlling digestion;
(*b*) Observing the progress of digestion;
(*c*) Controlling the temperature;
(*d*) Collecting and utilizing the products of digestion;
(*e*) Removing digested sludge;
(*f*) Handling the overflow of displaced liquor;
(*g*) Admixture of new and old sludges;
(*h*) Handling and overcoming scum troubles; and
(*i*) Moving sludge from one part of a tank to another.

Volume.—In the older plants it was impossible to achieve satisfactory results without providing much sludge storage. This, in general, has been due to the method of operation and not to any unusual characteristic of the sewage or its organic solid content.

As Mr. Gould has shown,* the first step in the digestion cycle is an acid fermentation, which is followed to completion later by an alkaline series. Modern research has clearly shown that if a tank is filled with sludge undergoing acid decomposition, the entire tank will be acid. Unless this is overcome artificially, sufficient time must be allowed for the natural reaction to take place. This means large volumes, large tanks, and large area consumption. Dr. Willem Rudolfs and others have shown, however, that if a fresh sludge is added daily to a tank filled with a mass already alkaline, the acidity created by the new raw sludge will not upset the reaction of the tank, and alkaline digestion will continue. This daily proportion has been roughly placed at from 2 to 3% of the total. Sludge that is not more than 24 hours old, preferably 8 to 16 hours, is defined as fresh, raw sludge. To obtain these results is not difficult and requires little technical skill. In plants that require only the part time of one ordinary laborer per day, successful sludge digestion has been accomplished over periods of years. Naturally warm climates are more favorable to continued digestion, but records show equally satisfactory results in cold northern sections.

Mr. Gould discusses* the modern method of expressing the environment for successful digestion—the hydrogen-ion concentration, expressed as pH-values. Apparently, a misunderstanding is current. Several investigators have found that the pH-range for best results varied from 7.2 to 7.6, others from 7.4 to 7.6, or even more. Some operators have taken this to mean that it is necessary to maintain a pH-value of at least 7.2 to 7.4. This is not so. If 7.2 to 7.4 is found to be the best range for some plant, no one should infer or claim that it should be applied arbitrarily to every case. Intensities in pH-values of 7.1 and even in those as low as 6.8 have produced good

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2658.

results. Unless, it can be proved that a certain pH-value is necessary to produce good digestion, no extra money should be spent on re-agents to procure a given degree of alkalinity. Only experimental work can determine the most economical limit.

TABLE 2.—RANGE OF pH-VALUES MAINTAINED AT VARIOUS SEPARATE SLUDGE DIGESTION PLANTS.

Location.	pH-Value.	
	Liquid.	Sludge.
Hartford, Wis.	6.8 to 7.0	6.7 to 8.1
Kiel, Wis.	.....	7.1 to 7.4
Sheboygan, Wis.	.....	7.4
Antigo, Wis.	7.2	7.4 to 7.6
Ridgewood, N. J.	.....	7.0 to 7.4
Storm Lake, Iowa.	.....	7.2 to 7.6
Sioux Falls, S. Dak.	.....	7.3

The fluctuation in necessary pH-values in different plants is shown in Table 2. These plants vary in size, characteristics of sludge, locality, operation, etc.

The volume of sludge digestion tanks is a matter of extreme importance. The customary unit of "cubic feet per capita" has led to misunderstanding, because it has not always been clear whether it was based on present, maximum, future, or some intermediate population.

Authorities now agree that a better way is to record the effective capacity on the basis of the solids received in the tank, that is, the cubic content of the digestion tank from the bottom to a point 1 ft. below the over-flow. This point is taken to prevent the discharge of sludge through the over-flow pipe.

As temperature is important for procuring year-round digestion, a digestion tank for warm climates would require less volume than one for a cold climate. If the warm temperature favors uninterrupted digestion throughout the year, aside from all other considerations, this would warrant the use of a small volume per capita. Conversely, in a colder climate for months digestion is inhibited, and the tank functions as a storage tank only; therefore, sufficient volume must be provided to accommodate the daily sludge contribution for the dead period. No arbitrary scale of the proper volumes per capita can be stated, as other elements besides temperature influence the speed of digestion. The maintenance of a favorable alkaline environment, the characteristics of the sludge itself, and the character of the solids, all tend to influence the digestion time and the volume required. Certain industrial wastes contribute such a large extra load of solids producing acid products as to require from three to four times as much volume per capita as normal sewage sludge.

Tanks without artificial temperature regulation, and those in which the acid step is not controlled by proper additions of sludge or the use of corrective agents, have a lag in digestion because during the period of acid

fermentation, there is no digestion. This must be provided for in extra volume.

Knowing the optimum temperature and the correct proportioning of raw and digested sludge, an engineer can design tanks that will produce well-digested sludge throughout the year with a minimum of capacity.

Design data of fifty-one plants constructed in all parts of the United States since 1923 show that the average per capita volume in the digestion tanks is 2.87 cu. ft. The range is from 1.15 to 12.0 cu. ft., the latter being for a plant having a large proportion of industrial wastes.

Shape.—Assuming that control of temperature and reaction is to be provided, the next consideration is the shape of the tanks.

Construction cost enters into this largely. Simplification by using straight walls, omitting baffles and intricate sections decreases the initial cost. The simplest form is a vertical cylinder, with no internal walls or baffles and a relatively flat bottom. This simplifies form work and reduces the amount of reinforcing steel, at the same time producing the strongest possible wall. It assures speed of construction and avoids the occurrence of pockets to trap gas.

Practically all the tanks built in the United States and Canada since 1924 have been vertical cylinders. Some have flat bottoms, cleaned mechanically; others have cone bottoms, depending on hydrostatic evacuation of the sludge. Some have flat roofs carried from the tank walls; the writer agrees with Mr. Gould* that domed roofs are not only difficult and uneconomical to build, but introduce an element of danger in allowing the accumulation of large volumes of gas which may become extremely explosive if air is admitted to the tank. In modern tanks, with the liquid level maintained at the bottom surface of the tank cover, the only available gas collection space is the small dome and the danger is negligible.

The number of tanks required is a matter of argument. A study of fifty-one plants constructed since 1923 shows that in all except the largest the number is usually two. As tanks up to 85 ft. in diameter have been in operation for some time, and those of larger diameter may prove equally practical, there is no mechanical reason for providing a large number. Doubtless, economy is best served by fewer and larger tanks. As digestion tanks are provided for the sole purpose of allowing organic solids to undergo a natural process of decomposition, a larger capacity than is required at the start is no detriment. Therefore, fluctuating population is not a good argument for a large number of small tanks.

As no definite advantage has been shown in progressive digestion, that is, transferring sludge from one tank to another, over controlled digestion in one tank, no economic purpose can be served by more than two or three units. Furthermore, in heating one or two large units, the heat loss due to the addition of daily sludge will be less than in smaller units; therefore, it should be possible to maintain more equal temperature. Less heat also will radiate, as the aggregate wall area of the larger units will be less. Furthermore, in places where mechanical units are used for distributing and collecting sludge and releasing gas, a few large units will cost less initially and less to operate than a larger number of small units.

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2661.

Operating Devices.—The next considerations are the means for controlling digestion and temperature, for collecting and utilizing products of digestion, for eliminating scum, for mixing new and old sludge, for removing displaced liquor, etc.

As one of the products of digestion is gas, which when disseminated in quantities in the atmosphere is productive of obnoxious odors, the modern idea, as expressed by Mr. Gould,* is to cover the tanks. This is true regardless of locality. Tanks operating, for instance, in the South, would not normally require covers, because there is no need for heating the tanks, and, therefore, for collecting the gas to utilize its heat value; but from the standpoint of control of odor, an uncovered tank is never a certainty, therefore, even in warm climates, covering is frequently advisable, especially where plants are close to residences. The gas may be burned merely to provide against odor, or may be used as a source of heat or power as required. In any section where a period of cold weather may reduce the tank temperature to the danger point, heated and covered tanks are proper.

Research and observation show definitely that a separate sludge digestion plant with modern digestion units, controlled temperature, and reaction, treating an average municipal sewage, will produce sufficient gas to maintain the optimum digestion temperature throughout the year, and, in most cases, will provide heat for the plant buildings and frequently for the glass-covered sludge beds as well. This economy is provided at no cost.

At Antigo, Wis., the cost of electrical current to operate all units of the plant for the eighteen months beginning July 1, 1927, averaged $30 per month. The total production of gas was 2 300 000 cu. ft. Assuming that this gas had been sold at a rate of 30 cents per 1 000 cu. ft. (a low rate for a high-grade gas), the total revenue would have been $690. On this basis the gas produced would have offset the entire cost of electric current for the period. As a matter of fact, the gas was used to heat the digester and the plant buildings; if the required gas had been purchased locally, it would have cost $215.25 per month. If soft coal had been used, it would have cost more than $1 000. With an ordinary circular tank, the apparatus is moderate in cost and simple to operate and control.

Mr. Gould refers* to the formation of scum in digestion tanks. Even with reaction control scum may form, but, as he states, it is more likely to occur within correct reactions. Scum that forms in a tank with gas collection domes in the cover prevents the free and even discharge of the gas and may interfere with the uniformity of flow to the heating system, requiring a holder of considerable volume to equalize the flow. A tank equipped with a revolving mechanism to break down the scum continually will provide an even flow of gas and obviate the gas-holder except in very large plants, where it is desired to store the surplus gas during periods of warm weather. A mechanism of this type is shown in Fig. 3.†

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2661.
† *Loc. cit.*, p. 2663.

A digestion tank, particularly a closed one, should be kept full of sludge and liquid at all times. Withdrawal of sludge to the drying beds will cause the level of the liquid in the tank to lower unless the period of withdrawal is timed to correspond with the period of sludge addition, and this is a simple accomplishment. If the over-flow pipe is trapped and baffled properly, there will be no chance for air to enter the tank during withdrawal periods, and little if any chance for solids to pass out of the tank.

The liquid displaced from the digestion tank is naturally equal in volume to the amount of sludge put in the tank, except during the period when sludge is being withdrawn and admitted at the same time. A large number of plants return this displaced liquid to the inlet side of the sedimentation units. Most of the settleable solids that might pass out of the digestion tank with this liquid are re-settled and returned to the digester, and the volume of this liquid displaced in any one day, as compared to the total volume of sewage flowing through the sedimentation unit during the same day, is so small that, in general, this return of displaced liquid would not upset the operation of the sedimentation tank. However, several State Departments of Health have issued rules requiring the treatment of this displaced liquid separately. This usually entails only the construction of a small sand filter.

Digestion tanks may be heated in several ways. The simplest and cheapest is to circulate the gas-heated water through the digestion tank by a coil of pipe on the walls. The first form used was similar to the familiar factory heating coil with several pipes running to manifolds at each end. This was found to give a little difficulty in heat loss and ineffective heating of all parts of the coil, so that the favored type is a spiral running completely around the inside face, starting about 2 ft. from the bottom of the tank and having four to six complete turns. As the hot water must go in one end and out of the other, there can be no tendency to clogging or sluggishness of flow.

Observations have shown that the initial temperature of the hot water should be sufficient to transfer approximately 20° Fahr. to the sludge to maintain the optimum range of 75 to 80° Fahr. There is, however, a danger point to be avoided here. If the initial temperature is too high, the sludge may dry and cake too rapidly around the sludge pipes. This will make them ineffective as heating units, and cause a continuing increase in the initial temperature of the water. By experience even in the extreme northern section of the country (Antigo, Wis.), an initial temperature of 120° Fahr. will allow continued satisfactory operation and no caking will result.

Other means of applying heat to the sludge have been suggested, such as circulating hot water through it, depending on the convection currents to warm the entire mass. The writer believes that this method will entail greater heat losses than the pipe-coil method, as the hot water will naturally rise to the surface and pass from the tank with the over-flow; and that in passing upward it will not diffuse through the entire mass, but will flow upward in streams through the sludge, heating only the portion directly adjacent to it in its travel.

When the sludge is completely digested, it should be removed to the drying beds, thus freeing space for new sludge. If hydrostatic evacuation is relied upon solely to move the sludge, this evacuation will not be complete and certain volumes will remain upon the walls and bottoms, caking and eventually reducing the effective digestion space. True, this may be prevented by sprays of water or air operated at the time sludge is withdrawn, or by hand-squeegeeing the slopes; but there will be no guaranty of complete evacuation. The addition of water increases the moisture of the sludge, the driving period, and the volume of over-flow from the top of the tank. Air is apt to cause flotation of solids, forming surface scum. Hand-squeegeeing introduces an element of cost and is not entirely satisfactory. Any of these three methods requires eventual dewatering of the tank and careful cleaning of the surfaces. By means of a mechanism (see Fig. 3), complete evacuation of the sludge may be obtained.

If the sludge is allowed to compact in the bottom of the tank, without any attempt at stirring to procure an admixture between the old, partly digested sludge and the new raw sludge, digestion will be stratified. By providing a means for gently stirring the mass, keeping it porous, and allowing the continuous upflow of the gas in small bubbles, the formation of thick surface scums, which is so common in many tanks, is prevented.

One of the recognized features of modern plant operation is the control of alkalinity in the digestion tanks. This usually consists of adding sufficient lime to the sludge to overcome the acidity. It is simpler to add the small quantity of lime required directly to the daily volume of sludge than to the entire tank volume. Hence, the tank which permits this will be an economical unit in operation. In several cases in the writer's experience with plants of this type, when no attempt at reaction control was made at the start, it has been found that after a period the entire tank contents were acid. It was then necessary to add sufficient lime to the tank to bring the entire mass to the alkaline sludge and after that to add sufficient lime each day to the new sludge admitted to maintain this alkalinity until tests showed that the tank was entirely in alkaline balance and the daily addition of new sludge would not upset it. This was a very simple operation with the mechanical type of digester. The lime for correcting the tank mass was simply dumped in at the top of the tank and the mechanism worked it down into the sludge; the lime required for the correction of the daily volume was added in the open flume where the sludge was received from the pump at the top of the tank. In one case (Lakeland, Fla.), this entire operation occupied a period of about six weeks, and since then the tank has operated most satisfactorily, without odors and without the addition of any lime for more than two years.

Distribution of sludge to all parts of the area of the tank is important to procure the prompt seeding of the raw sludge and to insure complete digestion. The mechanical type of digester provides this, as the revolving mechanism tends to distribute the sludge thoroughly to all points in the tank.

In the writer's opinion, the entire question of design is most important, and engineers in general cannot give too much time to the consideration

of every factor in the endeavor to produce plants which will function in a satisfactory manner. The trend of thought as regards sewage treatment works is rapidly changing for the better, and municipalities now are looking upon them as important utilities and are willing to spend the sums necessary for proper construction. The modern sewage treatment plant using the proved means for digesting sludge may be constructed with safety, in sections close to residential districts or near main highways without being considered eyesores or sources of nuisance.

JERRY DONOHUE,* ESQ. (by letter).†—The advance in the art of sewage disposal actuated by a public demand for better treatment plants has been most pronounced in the period since 1900 and indications point to still further perfection in the science in the future. Problems of sewage disposal are now public questions which demand study and analysis not only by municipal officials but by the taxpayers who are called upon to pay the bill and who are demanding better treatment plants. They insist that industries stop the pollution of streams by constructing sewage plants or by combining forces with the municipalities in the study of the problem.

As has been so clearly emphasized by the author,‡ separate digestion and the design of better digestion tanks have been brought about by scientific research, and the development of this method is merely a link in the chain of progress made possible by a working knowledge of the plants which have demonstrated these improvements in successful operation. Some of the advantages of separate sludge digestion may be classified as follows.

First.—Simplicity of Construction.—A separate sludge digestion plant is built of simple concrete structures. The digestion tanks are constructed above the water-table, and do not entail any unusual construction cost or expensive excavation.

Second.—This type of plant is operated mechanically. This is a decided advantage to the smaller communities, because it compels careful operation. The old type of sewage disposal plant, when once installed, was neglected and ignored as the municipal authorities paid little attention to operation. The mechanically operated plant requires attention; therefore, the process of sewage purification will go on more efficiently and better results will be obtained.

Third.—Flexibility as to Expansion.—Either the sedimentation unit or the digestion unit may be enlarged and thus maintain a balance between them. If conditions should arise which would require an expansion of the digestion compartment, this can be accomplished by merely adding another digestion tank.

Fourth.—The separate sludge-digestion type of plant is better adapted to the collection of gases than most other types, as the gases can be trapped in the digestion tank and conducted to a gas burner, thus utilizing a waste product for heat and power and increasing the efficiency of the plant.

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* Cons. Engr. (The Jerry Donohue Eng. Co.), Sheboygan, Wis.

† Received by the Secretary, February 15, 1929.

‡ *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2657.

Fifth.—The period of digestion made possible by the application of heat to the sludge may be extended to a full twelve months, whereas in most types of plants the digestion process stops just as soon as the temperature drops below 50° Fahr. By adding heat to the sludge, the period of digestion can be extended and accelerated, resulting in a saving in capacity. In the middle western and northern climates where the temperature ranges from 30° below zero to 90° or 100° Fahr., above zero, the extension of the period is a decided factor in an economical selection of the type of plant best adapted to the conditions.

The cost of mechanical operation in separate sludge digestion plants is very small. The Hartford, Wis., plant was the first of this kind in the State of Wisconsin. The cost of power to operate this plant is $15 per month, or about 50 cents per day. This includes all power necessary for motor operation, pumping of sludge, and the lighting necessary in the building.

The total annual cost of operation of the plant at Antigo, Wis., including power, light, chemicals, etc., is 64½ cents per capita. From this cost, however, should be deducted the value of the reclaimed gas, which is utilized in heating the sludge in the digester. If the value of this gas is credited at the rate charged by the local utility company, it would reduce the cost of operation to about 18 cents per capita.

Separate sludge digestion tanks should be built with tight covers to prevent odors from escaping into the air and to permit the efficient collection of gases. The type of cover usually recommended is a concrete slab constructed so as to permit it to expand and contract with the temperature. This concrete slab, which is supported by the circular side walls and through the center by the same truss that carries the operating mechanism that agitates the sludge, rests upon a mastic joint extending the full thickness of the supporting wall. In the center of the supporting wall, the joint is keyed into the concrete. The depth of the key provides space for a crimped copper strip which extends around the full circumference. The joint is filled completely with asphalt which allows the copper strip to elongate under the stress of the moving concrete top, the copper being firmly held at both ends by its embedment in the concrete roof slab on the top and the concrete wall on the bottom. This form of joint makes the digester both water-tight and gas-tight.

When sludge is withdrawn from the digestion tank, the level of the liquid in this tank is lowered. This raises the question as to the advisability of having a gas container in series with the gas line so that after sludge is withdrawn, gas will follow back into the digestion tank to take the place of the liquid that has been removed. A gas container, with a capacity of one day's or two day's supply, would have the further advantage of stabilizing the flow of gas to the burners and of supplying gas during the time that sludge is being withdrawn, or while repairs are being made to the gas dome or digester.

It is predicted that the development of the next few years will include a combined type of plant of activated sludge with separate sludge digestion, using the better features of each type. This combination will enable smaller cities to consider activated sludge treatment, with the reduc-

tion of the sludge by separate digestion. This will largely solve the sludge problem for communities that experience difficulty in disposing of their sludge, for by digestion the volume is reduced 40%, and the residual sludge dries readily and makes a good fertilizer.

The Antigo plant seems to be producing approximately 1 cu. ft. of gas per capita per day. Assuming a conservative estimate of 0.8 cu. ft. per capita per day, the annual value of the gas based on the local utility rate would be about $0.45 per capita. The heating value of the gas, as determined by analysis, averaged 550 B. t. u. per cu. ft., so a recovery of more than 400 B. t. u. per capita per day can be depended upon. This, reduced to coal at 12 000 B. t. u. per lb., is equal to 6 tons per year per 1 000 population.

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PAPERS AND DISCUSSIONS

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ELASTIC EQUILIBRIUM IN THE THEORY OF STRUCTURES

Discussion*

BY MESSRS. JACOB FELD, F. B. SEELY, W. J. PUTNAM, and W. L. SCHWALBE, NORMAN LITCHFIELD, and E. R. HILL.

JACOB FELD,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The author opens up a subject which can fill many volumes, and which is the very basis of structural design, and then discusses in detail one phase of the subject, namely, the true action of two types of structural members acting as beams.

The great majority of present-day designs are based on formulas which are known to be purely approximate. The unknown as well as the omitted factors in determining maximum stress, are provided for by the use of a unit stress which is admittedly much lower than that at which members may fail. This method, of course, cannot be classed as the scientific procedure and has as its virtue only one fact, that a very small percentage of structures so designed have been known to fail.

It seems to the writer that three questions must be completely answered before a scientific design procedure can be expected: (1) What type of stress causes failure; (2) what is the actual distribution of stress through a section; and (3) what re-distribution of stresses occurs in a section when one fiber is stressed beyond its elastic limit. The writer makes no attempt to discuss these points.

Of course, it is fairly simple to determine all the possible loading conditions and, if time is available, to investigate stresses resulting from every possible loading; but the maximum stress which exists under any one condition of loads and the true excess of strength before failure occurs cannot be determined so easily. If failure has occurred, as in a test specimen, the type

* This discussion (of the paper by H. S. Richmond, Esq., published in January, 1929, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, January 16, 1929.

of stress causing failure often can be determined from an inspection of the fracture.

In general, the actual stresses, both theoretical and experimental, are greater than the apparent stresses. The true or equivalent stresses are a refinement over the apparent stresses caused by the addition of combined normal and shear stresses as opposed to normal stresses only.

Experimental determination of stresses in various members acting in bending has shown greater stresses than those calculated by the ordinary empirical methods. However, the measured values agree closely with those computed by the ordinary elastic theory if due allowance is made for lateral strain (Poisson's ratio).*

In dealing with beams under bending and torsion the author has introduced a subject in which the ordinary analysis gives results that do not agree with experimental data. It is only necessary to point out the great variation between analytical and experimental results for the effective polar moment of inertias as tabulated in a report of tests on the torsional strength of steel I-sections.†

The effect of torsion on beams in ordinary building construction, although never considered in the average design, is marked by unsightly results in many structures. This is especially apparent in spandrel beams in concrete structures which very often show cracks on the exterior face at approximately the quarter points of the span. Similar torsional strains are visible on steel buildings, in which case the brick or stone facing shows open joints.

However, in the type of frames for which the author develops his theory, the effect of torsion due to eccentricity of loading can be included in the analysis. It must be remembered, however, that experimental results also show a variation of shear stress across the depth of the web, the shearing stress being much greater in that part of the web nearest the loaded flange than near the unloaded flange. Therefore, if a beam first has an unequal unit shear stress in the web area, even with axial load, the condition of neutral elastic equilibrium used by the author as a criterion for the strength of a beam, must take into account the combination of shear and tension or compression at the junction of the web and flange areas near the loaded flange.‡

With reference to the author's discussion of the proper use of channel members as beams,§ it is proper that several contributions to the subject be enumerated. In a thesis presented at the University of Zurich, in 1920, H. Schwyzer proved mathematically that in order to produce bending without twisting, a channel section must be loaded by forces parallel to the web and a distance, $\frac{3\,b}{6+\frac{F_1}{F_2}}$, from the back of the channel. The factor, b, is the width of the flange, F_1 is the area of the web, and F_2 is the area of one flange.

* "The Strength of I-Beams and Girders," by Herbert F. Moore and W. M. Wilson, *Bulletin No. 86*, Eng. Experiment Station, University of Illinois, 1916.

† *Bulletin 4*, Univ. of Toronto, 1924.

‡ "Observed Stress Distribution in Web of I-Beam," by O. H. Basquin, *Engineering News-Record*, June 3, 1926.

§ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 8.

A detailed method for the calculation of torsional stresses has been developed by Dr. A. Eggenschwyler.* Other contributions to the subject are those of Maillart† and Zimmerman.‡ These men have checked their analyses against the experimental work of Bach who found among other things that the longitudinal stress at the connection of the web and flange of channel under load is 36% greater on the compression side, and 53% greater on the tension side, than the values given by the bending formula.

If the author suggests that structures be designed for the maximum allowable stress without a factor of safety, he must give methods of analysis which will determine the maximum existing stress under any given loads. He is to be commended for his addition to the ultimate goal of all design analysis.

F. B. SEELY,§ W. J. PUTNAM§ and W. L. SCHWALBE,§ ESQS. (by letter).‖—The author points out¶ that when a channel is used as a beam without lateral restraint the stresses set up in it will not be in accordance with the simple flexure formula, $f = \frac{M\,h}{2\,I_h}$, unless the "position of the loading system is that of a plane which lies parallel to the web and a distance back from the center of the web equal to $\frac{U\,b}{2\,I_h}$."

It has been very widely assumed, however, that the loads should be applied through the center of gravity of the sections in order to cause a channel to bend without twisting and to produce stresses in accordance with the simple flexure formula.

It may be worth while, therefore, to present experimental evidence of the fact that a channel must be loaded back of the back of the channel (not through the center of gravity of the cross-sections) in order to cause bending, without twisting, as is assumed in the simple theory of beams.

The writers have made bending tests** of a number of rolled steel channels, varying in depth from 4 to 15 in., in which the positions of the loads that caused bending in accordance with the flexure formula were determined. The main purpose of the tests, however, was to determine the additional longitudinal stress in a channel beam when loaded so that twisting accompanies bending. This additional longitudinal stress, for some channels, when the load acts through the center of gravity of the section may be more than one-half the maximum stress caused by the vertical bending moment.

The name, "shear-center", has been used to denote the point on the axis of symmetry of the channel through which the plane of loads must pass to produce simple bending (without twisting), and its location and significance seem to have been brought first to the attention of engineers by R. Maillart†

* *Eisenbau*, 1921; *Bauingenieur*, 1922.
† *Schweizerische Bauzeitung*, April 30, 1921.
‡ *Bauingenieur*, April 30, 1921.
§ Professors, Dept. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.
‖ Received by the Secretary, January 25, 1929.
¶ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 9.
** In the Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

in 1921, and by A. Eggenschwyler* and A. Ostenfeld† in 1924. In 1925, K. Huber‡ obtained experimental results on a rolled steel channel 10.3 in. (26.14 cm.) deep, tested as a simple beam on a span of 20.8 ft. (5.8 m.), and subjected to a concentrated load of 5 500 lb. (2 500 kg.) at mid-span; the position of the "shear-center," as found from the test results was in substantial agreement with the calculated position.

In order to determine the "shear-centers" of the channels tested at the University of Illinois, each channel was tested as a horizontal cantilever beam with a vertical load consisting of a suspended weight applied at the free end. Some of the smaller channels were embedded at one end in a large cube of concrete and this cube was held on the weighing table of a testing machine by applying a load to the top face of the cube by means of the movable head of the testing machine. The large channels were tested as double cantilevers. In Fig. 8, a 15-in., 33.9-lb. channel is shown with a load of 5 600 lb. applied through the shear-center at the end of a 10-ft. cantilever. In Fig. 9, the same load is applied at a point on the flange directly above the center of gravity of the end section when the section is not twisted. The beam was balanced on a knife-edge on the weighing table of a testing machine and equal weights were applied to the two free ends. The central section of the channel was held fixed in position by a light load applied through the movable head of the testing machine to the top flange, and by small horizontal jacks on each side of the top and bottom flanges. In addition, the outstanding flanges at the central section were supported by a vertical stiffener extending between the flanges.

The angle of twist was measured at six or seven sections along the length of the beam for each of several lateral positions of the load at the end. The load was applied at two or three points on each side of the back. An angle was clamped to the back at the free end of the beam so that the load could be applied back of the back of the channel. (See Fig. 8.)

The angle of twist was measured by means of a level bar used on a 20-in. gauge length. For this purpose bars slightly more than 20 in. long were clamped to the top flange of the channel at several sections along the length of the beam, and the change in inclination of these bars was measured on a 20-in. gauge length; the change in elevation of one end of the gauge length with respect to the other was measured to 0.001 in. by adjusting a micrometer screw at one end of the level bar. The bubble in the level bar was a 20-sec. bubble, which was amply sensitive.

The values of the angles of twist at three sections for five different lateral positions of the load on a 6-in., 8.2-lb. channel are given in Fig. 10. (One point on Curve A — at $+$ 0.5 in. $=$ 0.0735 radians — has been omitted.) The diagram shows clearly that this channel beam twists, as it bends, when the load is applied at any lateral position, except that at the distance of 6.60 $+$ in., approximately, from the center of the web. The calculated value for the distance of this "shear-center" from the center of the web, according

* *Schweizerische Bauzeitung*, Vol. 83, pp. 259-262 (1924).

† "Teknisk Elästicitetslære," Fourth Edition (1924).

‡ *Der Bauingenieur*, Vol. 6, p. 458 (1925).

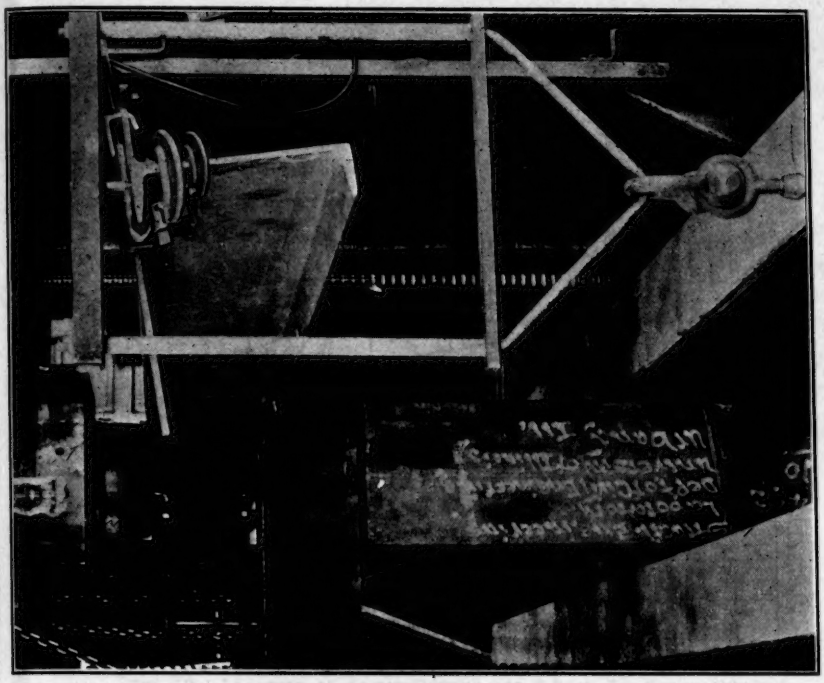


FIG 9.—VIEW OF CHANNEL WITH LOAD APPLIED AT POINT ON FLANGE DIRECTLY ABOVE CENTER OF GRAVITY OF END SECTION WHEN SECTION IS NOT TWISTED.

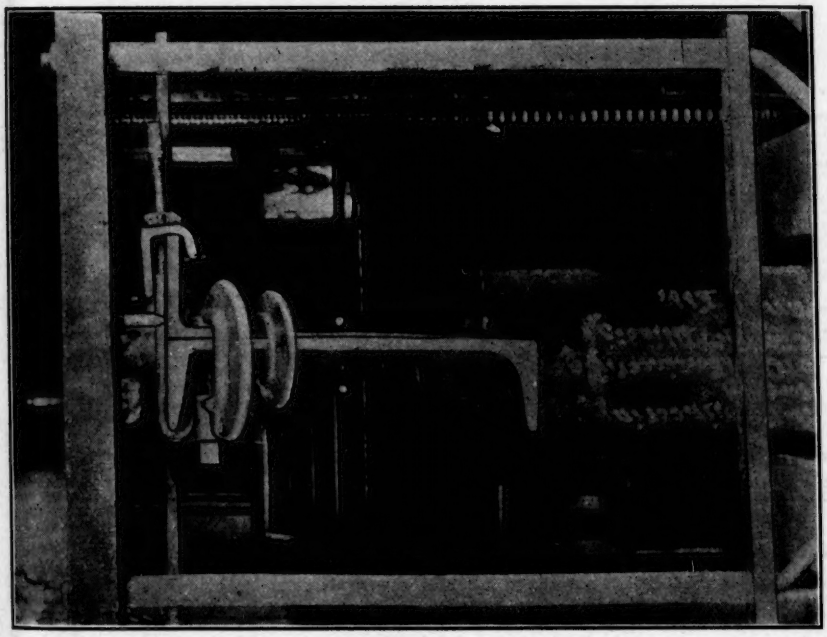
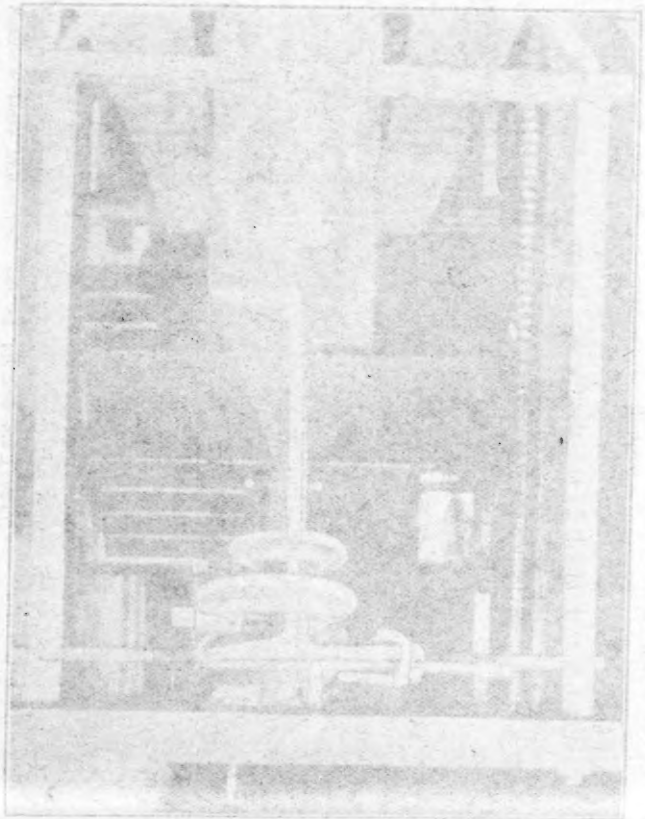
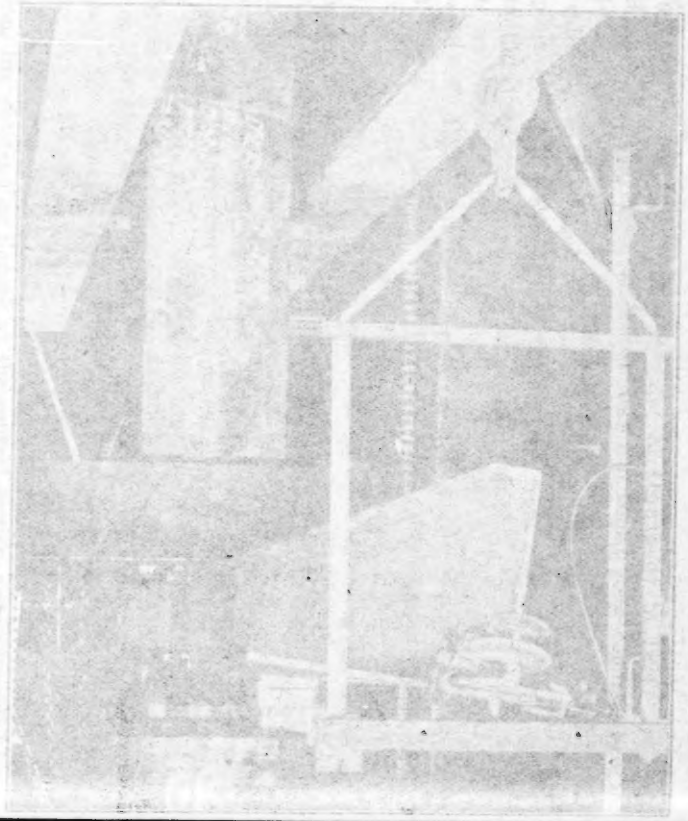


FIG. 8.—VIEW OF CHANNEL WITH LOAD APPLIED THROUGH SHEAR CENTER AT END OF CANTILEVER.





Longitudinal Stress in Thousands of Pounds per Square Inch

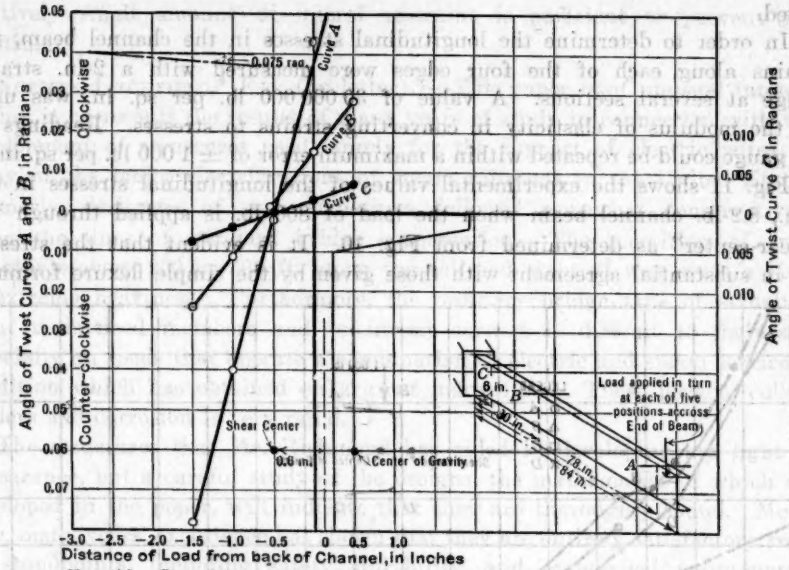


FIG. 10.—LOCATION OF SHEAR-CENTER FOR 6-INCH, 8.2-POUND CHANNEL.

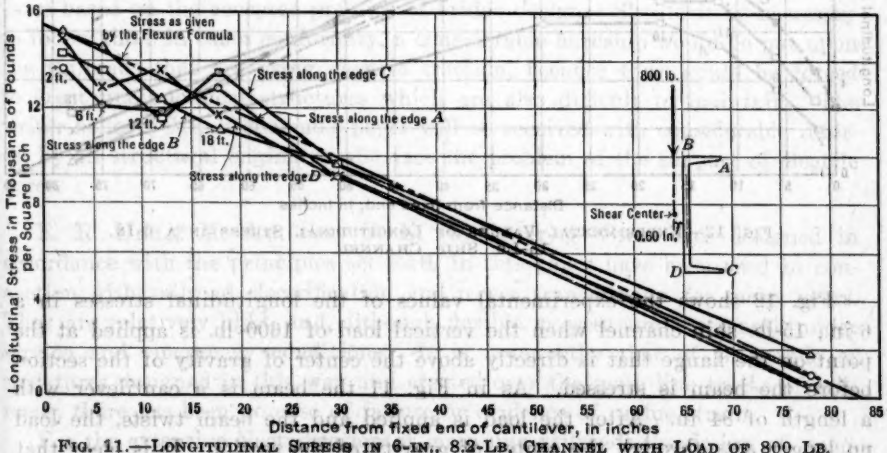

Longitudinal Stress in Thousands of Pounds per Square Inch
Stress as given by the Flexure Formula
Stress along the edge C
Stress along the edge A
Stress along the edge B
Stress along the edge D
2 ft.
6 ft.
12 ft.
18 ft.
800 lb.
Shear Center
0.60 in.
Distance from fixed end of cantilever, in inches

FIG. 11.—LONGITUDINAL STRESS IN 6-IN., 8.2-LB. CHANNEL WITH LOAD OF 800 LB. APPLIED THROUGH THE SHEAR CENTER AT END OF CANTILEVER HAVING A LENGTH OF 84 IN.

to the expression, $\frac{U\,b}{2\,I_h}$, is 0.618 in. A satisfactory agreement between experimental and calculated results was obtained for the other channels tested.

In order to determine the longitudinal stresses in the channel beam, the strains along each of the four edges were measured with a 2-in. strain-gauge at several sections. A value of 30 000 000 lb. per sq. in. was used for the modulus of elasticity in converting strains to stresses. Readings of the gauge could be repeated within a maximum error of ± 1 000 lb. per sq. in.

Fig. 11 shows the experimental values of the longitudinal stresses in the 6-in., 8.2-lb. channel beam when the load of 800 lb. is applied through the "shear-center" as determined from Fig. 10. It is evident that the stresses are in substantial agreement with those given by the simple flexure formula.

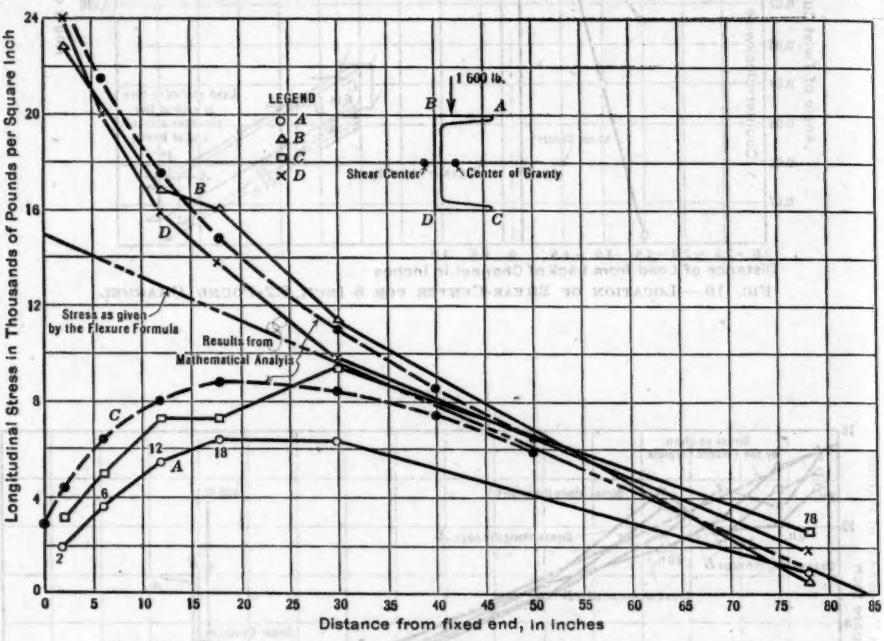


FIG. 12—EXPERIMENTAL VALUES OF LONGITUDINAL STRESS IN A 6-IN. 15-LB. SHIP CHANNEL.

Fig. 12 shows the experimental values of the longitudinal stresses in a 6-in., 15-lb. ship channel when the vertical load of 1600 lb. is applied at the point on the flange that is directly above the center of gravity of the section before the beam is stressed. As in Fig. 11 the beam is a cantilever with a length of 84 in. After the load is applied and the beam twists, the load no longer acts through the center of gravity of the section. It is clear that the twisting of the beam causes the stresses along two edges near the fixed section to be greatly in excess of those given by the simple flexure formula, and those along the other two edges to be much less than those found from the flexure formula.

If, however, a channel is used where it has lateral support so that it is restrained from twisting as it bends, this additional longitudinal stress does not develop; furthermore, due to the small torsional stiffness of channels, a relatively small amount of lateral restraint is sufficient to prevent the twisting.

NORMAN LITCHFIELD,* ESQ. (by letter).†—This paper is of unusual interest in that it represents the results of many years of study in connection with the development of structures used largely for the support of electric wires on many of the major electrifications of steam railroads in the United States. Formerly, designers of such structures followed somewhat conservatively along the lines of standard bridge construction. The repetition of such structures every 250 or 300 ft. along the railroad right of way gives a vista of extreme heaviness. Furthermore, the ordinary bridge type of structure with its latticed members and re-entrant corners is difficult to maintain, especially on roads that operate a combination of electric and steam service, a condition which has obtained on a great many lines. These pockets collect cinders and corrosion is very rapid.

The structures that Mr. Richmond has aided in developing are light in appearance, but a careful study of the designs, the mathematics of which are developed in the paper, will indicate that they are thoroughly sound. Moreover, many years' experience has shown that they are entirely satisfactory from all standpoints, including safety, durability, and economical maintenance.

The subject which the author presents so well is one of increasing interest to railroad companies as congestion on many lines is gradually forcing more extended consideration of electrification. There has been an attempt to write into some specifications for structures of this character certain provisions based on the accepted practice of bridge design. Should it be necessary to follow these in exact conformity, a considerable hardship would be put upon the railroads that are using electric traction, because they would be forced to erect uneconomical structures which are also difficult to maintain. The writer believes Mr. Richmond's paper will be received with considerable interest by all structural engineers who face the problem of the support of electric wires.

E. R. HILL,‡ M. AM. SOC. C. E. (by letter).§—Structures designed in accordance with the principles set forth in this paper have been used in connection with railroad electrification and power transmission for many years. They are relatively light, and, although flexible, possess ample strength under service and emergency conditions. It is noteworthy that of thousands of structures designed in this way and erected over the main-line tracks of railroads, there has been no case of failure or evidence of undue strain.

For the extensive electrification of main-line railroads the design of structures for supporting the trolley wires and transmission and signal circuits

* With Gibbs & Hill, Cons. Engrs., New York, N. Y.
† Received by the Secretary, January 30, 1929.
‡ Cons. Engr. (Gibbs & Hill), New York, N. Y.
§ Received by the Secretary, February 20, 1929.

assumes a very important place. Such structures should involve minimum first cost and maintenance expense consistent with the satisfactory performance of the service required, and because of their extensive use they should be neat and attractive in appearance, avoiding the suggestion of a mass of steel over or along the tracks. A clear view of signals should be maintained. There is also a great duplication of structures of the same design, or similarity of type, and engineers can well afford to devote much detail study to an accurate analysis of the loads which must be carried, both normally and in emergencies. Where such determinations are made Mr. Richmond has found it practicable and entirely permissible to abandon certain of the empirical limitations that are generally imposed in the design of structures and develop methods of calculation and design especially applicable to these structures.

It is apparent that Mr. Richmond's theory and his mathematical development of it have been worked out very thoroughly. This paper should prove a valuable contribution to the art.

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PAPERS AND DISCUSSIONS

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STUDIES OF SHEAR IN REINFORCED CONCRETE BEAMS

Discussion*

BY MESSRS. EDWARD GODFREY, C. A. P. TURNER, H. E. ECKLES, and JACOB FELD.

EDWARD GODFREY,† M. AM. SOC. C. E. (by letter).‡—The writer is in complete accord with Professor Mylrea in his emphasis on the importance of re-examining "the basis of structural theory to check the assumptions on which it is founded."§ This is particularly true in any case in which the basic theory has been questioned and the arguments of the questioner have not been squarely met and answered.

One of the most vital questions in the field of reinforced concrete design is the stirrup or short shear member. It is of exceeding importance that free discussion be entered into and a conclusion reached as to the value of the stirrup—if it has any real value—and its proper place in design.

Professor Mylrea clearly considers of no importance the fact that stirrups in a whole beam have little or no stress. In the writer's judgment there is a real significance in this fact of test and analysis which is closely connected with the efficiency of stirrups as shear members. Experimental evidence shows that stirrups do not act until the concrete cracks. Analysis anticipates the same thing, for there can be no action in a homogeneous concrete beam that would elongate the beam in a vertical direction. Conceding the fact, then, that the stirrups in a reinforced concrete beam do not come into action until the beam cracks, the question arises as to what this involves. The unit shear allowed by some modern standards on the cross-section of a beam is a large fraction of the shear at which the first crack in the web takes place in test beams.

* This discussion (of the paper by T. D. Mylrea, M. Am. Soc. C. E., published in January, 1929, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Structural Engr., Pittsburgh, Pa.

‡ Received by the Secretary, January 21, 1929.

§ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 19.

Modern specifications allow a large share of the beam shear to be taken by the concrete. The margin between the shear allowed on the concrete web and that which cracks the concrete is small. In fact, it is not unusual to find beams in which the concrete web is cracked under safe load. What of the part of the shear allotted to the concrete web? A cracked web, unaided, could take no part of this shear. It will be thrown, therefore, into the stirrups, and it is largely in excess of the shear for which the stirrups are designed. Here is a condition, then, that can be expected in a standard design, namely, large over-stress in the stirrups.

In no other analysis of stress action is it assumed that any part of a structure is idle and inoperative until some other part fails. The writer does not believe this is sound analysis, particularly since there is a means of reinforcing a beam for shear that is free from this objection, both in its analysis and in its agreement with tests.

Professor Mylrea refers* to tests by A. N. Talbot, Past-President, Am. Soc. C. E., which proved that stirrups take no stress until the concrete cracks in shear. Tests by this same experimenter† prove that beams reinforced for shear by bending up the main reinforcing rods with a long flat bend, gave high stresses in the bent-up part of the rods. Furthermore, in the beams reinforced with stirrups, in an actual building, cracks were easily found in the concrete; whereas in beams reinforced for shear by bending up rods with a flat angle and crossing them over the support, no cracks could be found.

Analysis agrees perfectly with this, for both the shear deflection and the deflection due to bottom tension are accompanied by elongation in the beam in the diagonal direction in which the bent-up and anchored rods lie; and, since the rod is there to take the stress, cracks in the concrete are prevented.

A simple test and one that can be made in an office is as follows: Take a straight stick of wood about as long as a desk and about 1½ by 2 in. in cross-section. Fasten mandolin wires in screw-eyes as indicated in Fig. 11. Introduce small bridges to give a tension. Place this beam on two books and pick the wire. Then press down at the middle of the stick. The tone of the wire will immediately go up, proving the existence of tension in the direction in which the wire is stretched. It is in this position and direction that bent-up main rods are or should be placed, and every increment of load adds stress in these rods. Furthermore, failure of the beam in shear is impossible unless this rod severs or is pulled out of its anchorage. This cannot be said of any stirrup installation.

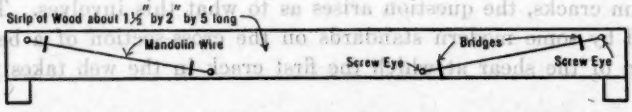

FIG. 11.—EXPERIMENT TO DETECT TENSION IN BENT-UP ROD.

It is manifest that a test made on this wooden beam with vertical wires on the sides, corresponding to stirrups, would not exhibit any increment of

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 21.
† *Bulletin No. 64*, Univ. of Illinois Eng. Experiment Station.

tension in the wires. Sharply inclined wires would show little if any added tension.

The author states* that the introduction of vertical stirrups would greatly delay failure, and this, he points out, is the function of vertical stirrups. Unless they can prevent failure entirely, and surely, some other form of shear reinforcement is a necessity. He assumes that the concrete beam cracks at an angle of 45° with the vertical, and the stress in a stirrup is determined on this assumption. The angle of this crack is the determining factor in finding the stress to be allotted to the stirrup, so there should be some good reason for the selection of 45 degrees. If the tangent of the angle is more than one, the stress in the stirrup is less in the inverse ratio. If that tangent is less than one, the stress allotted to the stirrup is greater.

There is no law that would compel the cracks to take an angle of 45 degrees. In fact, Fig. 6† shows cracks that are almost vertical. The writer has seen many actual beams and has read descriptions of many more that failed in wrecks by cracking at the line of support in practically a vertical plane. When actual beams fail at wall supports by breaking part of the concrete over the support itself, and when they carry with them the stirrups that were intended to prevent this failure, as the writer has observed to be the case, what ground can there be for the assumption that the concrete will crack some distance from the support and at an angle of 45° with the vertical? In Fig. 6, the cracks start at the support.

Continuous and restrained beams tested by Withey‡ are shown in many cases to have cracked exactly at the support. These beams had stirrups, but were not reinforced by bending and anchoring the main reinforcing rods as the writer would have done.

Assume a case in which a 45° line would cross three stirrups, and suppose, instead of cracking at 45°, the crack crossed only one stirrup. The stress in this stirrup would be three times that allotted to it by the designer. This increase of 200% in the stress in the stirrup would be merely the result of a crack in a concrete beam taking a different course from that assumed by the designer, and there is nothing inherent in the beam that would prevent it from cracking in that manner. In fact, the crack may miss the stirrups altogether, and a beam designed according to standard rules could fail in shear without in any manner disturbing or stressing the steel reinforcement placed in that beam to take the shear.

It is a fact that, between any two stirrups and between the last stirrup and the support, there is nothing to take the shear of a beam except the concrete and the horizontal steel. These would be there if no stirrups were used.

The author§ uses the truss analogy to explain and determine the stress in the stirrup. In the truss analysis the web members cut by any section, whether vertical or inclined at any angle, must be capable of taking all the

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 21.
† *Loc. cit.*, p. 37.
‡ *Bulletin No. 115*, Univ. of Wisconsin Eng. Experiment Station.
§ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 24.

shear of the truss. The analogy breaks down, therefore, when the inclination of the cut is arbitrarily assumed to be 45 degrees. The case then resolves itself into one of an arbitrary allotment of stress having no relation to steel truss analysis.

In another vital respect the truss analogy fails. In every truss, in steel design for example, every pound of stress in a web member must be traced through some adequate detail to an adequate resistance and, finally, to the reaction of the support. Professor Mylrea's analysis does not follow the stress of the stirrup beyond the end of the stirrup itself. The stirrup is said to be anchored. The nature of this anchorage corresponds to the end detail of a truss member. It should, of course, be capable of taking the full stress of the stirrup. The paper does not place much emphasis on the need of adequate anchorage, or of ways in which it can be obtained, and this should be a matter of searching inquiry, since it is a governing feature in the strength of the beam.

The anchorage at the bottom of the beam seems to be by a loop around one or more reinforcing rods. At the top of the beam it appears to be a short hook around a reinforcing rod or merely in the concrete. These are the standard and common end anchorages for stirrups.

No designer would seriously consider that a rod ½ in. square, for example, is adequately anchored for its safe load as a hanger if it is merely buried with a small hook about 2 in. long, or even looped about a rod close to the bottom of a beam with this same hook. Why should not the same principles apply in the case of a stirrup as would obtain in the case of a hanger on which an actual load is to be applied?

If it is assumed that the concrete itself furnishes the anchorage for the stirrup, how does that concrete deliver the shear load to the support where it must ultimately go? In a whole beam how does this avoid or lessen the diagonal tension in the concrete web? And if it does not accomplish this, why use the stirrup? In a beam, cracked at an angle of 45°, how can the concrete at the tip of the triangle in Fig. 1 (*h*)* be considered capable of anchoring the last stirrup cut by the 45° plane to the full value of that stirrup? Here, again, a designer would not seriously consider placing a capacity load on a hanger anchored in this manner.

By Professor Mylrea's reference† to Fig. 2 (*g*), it is evident that he considers the pull of a stirrup to go directly into the horizontal rods as a vertical load. This means that the load of the beam is hung as a vertical shear load on the horizontal rods. Analysis cannot justify this disposition of the shear load. Steel rods embedded in concrete cannot take any appreciable shear until the concrete fails in shear. The idea that steel rods as pegs could take shear at 10 000 to 12 000 lb. per sq. in. was discarded from specifications long ago. After the concrete fails in shear the horizontal rod would be in bending, and the capacity of a reinforced concrete beam would be little if dependence were placed on the bending strength of the rods to carry the shear to the supports.

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 21.
† *Loc. cit.*, p. 25.

While the author emphasizes* the necessity of anchoring the web reinforcement "at both top and bottom strongly enough to develop its full tensile strength", he does not explain the nature of that anchorage, nor how it could be effected at the top and bottom of the beam, which mark the extreme ends of the stirrups. *Technologic Paper No. 314*, U. S. Bureau of Standards, is referred to several times. This report describes tests in which the stirrups on shear members were fully anchored at the level of the top and bottom of the concrete web. In fact they were anchored into heavy beam heads which, in the investigation, were eliminated from consideration, so far as shear is concerned. For this reason the writer believes that those tests were of little or no value as teaching any lesson concerning reinforced concrete beams. Such beams, as constructed, do not carry with them large surplus depth that can be ignored in the calculation and yet be taken advantage of as anchorage space for stirrups.

Beam analysis must be based on beams as it is practicable to construct them. The tests described by the U. S. Bureau of Standards were on a comparatively thin concrete web fixed in a heavy concrete frame into which the web members were fully anchored and which was quite outside the area considered as concrete web. This does not correspond in any way with a reinforced concrete beam; hence analysis of a reinforced concrete beam cannot properly be based on results of these tests.

The author emphasizes† the importance of properly anchoring stirrups "at the ends". Also, he states that "sharp bends are taboo". He does not refer to the impossibility of fully anchoring a rod at its end. Unless a nut and steel plate is used, full anchorage at the end of a rod is not possible. A small hook either in concrete or around a main rod is taboo, because it involves a sharp bend. Anchorage in concrete requires a considerable part of the entire rod. Professor Mylrea states‡ that no form of hook will accomplish the anchoring of steel to develop the ultimate strength without appreciable slip. It would not be an unusual condition for a stirrup to require one-half the depth of the beam for the 100% anchorage that Professor Mylrea states is necessary. The web member is of zero length, and the truss analogy fails.

The author rightly emphasizes§ the importance of end anchorage of main reinforcing rods. In fact, with the top and bottom rods firmly anchored into the support, a large shear can be carried by the concrete even if it is cracked. The friction acting between the compression surfaces—if the crack is vertical—will take the shear component. It is allocating this shearing strength of the concrete web to stirrups, in interpretation of tests, that has given to the stirrup a place in design that its nature does not warrant.

If a continuous beam should crack a short distance from the support, namely, at the point of inflection, there would be no compression surface, because the moment is zero. A vertical crack at this location would create the dangerous condition of all the shear being taken by horizontal rods. If main

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 25.
† *Loc. cit.*, p. 33.
‡ *Loc. cit.*, p. 43.
§ *Loc. cit.*, p. 34.

reinforcing rods are bent up and crossed over the supports, as in Fig. 12, any section, cut at any angle, between the support and the bend in the rod, has a positive shear reinforcement that effectively prevents failure by direct tension in that reinforcement. The main rods are amply capable of "carrying" this tension and "delivering it" to its anchorage over the support where the reaction of the beam must be met.

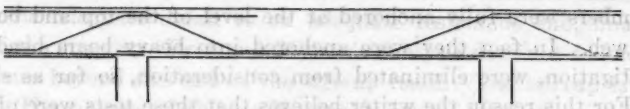

FIG. 12.—MAIN BARS ARRANGED TO RESIST SHEAR.

Professor Mylrea intimates that in future designs higher unit stresses will be used in concrete webs. The tests of the U. S. Bureau of Standards, to which he refers, showed cracks in the concrete webs at units of 200 to 300 lb. per sq. in. A concrete section nearly twice the nominal web area was actually in shear. These test beams, which were reinforced with stirrups with an efficiency that could not be approached in a practical design, failed in their concrete webs at about the allowed safe unit shear of standard specifications. They, therefore, did not even justify the units of those specifications.

C. A. P. TURNER,* M. AM. SOC. C. E. (by letter).†—The author attempts to analyze the web resistance of concrete beams which is a special case of the general problem.

Prismatic beams of homogeneous material were treated by Mariotte on the assumption ("on peut concervoir") that there were as many fibers compressed as stretched and, later, because the cross-section of the prism resisting horizontal shear was far greater than that resisting moment stresses, horizontal shear strain was neglected in the Bernoulli-Eulerian investigations which constitute the common theory of flexure as taught to-day. Mathematically considered, this neglect of shear strain is equivalent to treating the shearing modulus of the prism as infinite. This is the same as stating that a plane section normal to the axis remains normal to it after flexure and without distortion; that is, it still remains plane. This important normal relation is commonly overlooked in stating the hypotheses of the ordinary theory. An exact analysis of the trapezoidal components of deformation of the prism was thus developed, needing only the superposition thereon of the pure shear distortions to arrive at the exact theory of beams. The common theory is sufficiently approximate for the practical computation of the elastic behavior of simple prismatic beams having a length ten times their depth, but when the horizontal cross-section is reduced to a thin web, as in the broad flanged I-beam, or in the case of reinforced concrete beams, shear strain cannot be neglected for even approximate accuracy. For the restrained beam the greatest tensile fiber stress computed by the common theory is ordinarily from 20 to 30% greater than when computed by exact theory. Deflection in restrained beams is reduced by the interference of shear distortion. In the

* Cons. Engr., Minneapolis, Minn.

† Received by the Secretary January 21, 1929.

simple beam, on the contrary, the deflection is greater than that computed by the ordinary theory but the extreme fiber stress is the same at the section of maximum moment for symmetrical loading.

Weisbach suggested* the influence of pure horizontal shear on the position of the neutral plane in the homogeneous beam as one of the problems in developing the exact theory of flexure, but he neglected to solve the problem. Professor Karl Pearson proved that a neutral plane (defined as the locus of points at which tension and compression balance) does not in fact exist along the length at the gravity axis of the simple homogeneous beam. The writer has found† that the true neutral plane of the homogeneous beam curves downward toward the end of the beam approximately 0.09 of the depth by equilibrating—by the parallelogram of forces—the center of the shear stress area as the reaction to the fiber stress area.

The true location of the neutral plane is thus a curved surface of the same order as the slope curve. This is true also of the neutral plane of the reinforced concrete beam. The stress mechanism in the composite beam differs, however, from that in the homogeneous beam in that, although at the neutral plane there are right shearing stresses, they are oriented differently. The curves of horizontal shear are tangent to the neutral plane in the homogeneous beam, but they intersect the neutral plane of the composite reinforced concrete beam because the shear between the concrete and metal is equilibrated at the surface of the steel at the bottom of the beam. Accordingly, instead of curves of principal tension, curves of horizontal shear pair (symmetrically opposite, that is, with respect to the neutral plane) with curves of principal compression and the curves of tension in the concrete cut across the beam, assuming a position in the tension zone comparable to the curve of greatest shear in the homogeneous beam. With such a stress mechanism it is obvious that the neutral plane (in which the intensity of tension, compression, and shear are equal) is compressed along its length in proportion to the load applied as shown, but not recognized, in tests by the U. S. Bureau of Standards.

Accordingly, the neutral plane in the composite beam cannot be located by finding the locus of zero stress as may be done with a homogeneous beam. This radical difference in stress mechanism between the homogeneous and composite beam renders the investigation of the characteristics of the concrete beam by analogy to the homogeneous beam somewhat misleading. In vertical planes parallel to the axis, however, the curves of displacement are similar to those of the homogeneous beam, intersecting the neutral plane at a more acute angle than is the case with the homogeneous member.

There are two kinds of rotational distortion in a beam whether of reinforced concrete or homogeneous material. The rotational strain that is caused by the sharpness of curvature represented by trapezoidal deformation gives rise to equal diagonal tensions in the face of an originally square element in the tension zone and to equal diagonal compressions in an originally square element in the compression zone. The intensity of these diagonal stresses increases from zero at the neutral plane to a maximum at the outer fiber. This kind of strain causes cracking in the shallow beam toward the center.

* "Mechanics of Engineering", 1846.

† "Elasticity and Strength of Materials," Section II.

The second kind of rotational deformation is that of pure shear, or the tendency of the outer fibers to slide as two planks slide, one upon the other, when supported at their ends and loaded. This deformation produces a rhombic (pure shear) distortion, which is zero where the moment area is divided into equal segments (moment areas to the right and left of this section give rise to opposite shears which are equal to each other as action and reaction are equal) and increasing from that locus toward the end of the beam when simply supported. In rhombic distortion one diagonal of an originally square unstrained element is stretched; the other is compressed. Consequently, the maximum diagonal tension will occur at the location where these deformations, superimposed one upon the other, give the greatest diagonal strain. Because the rhombic deformation is proportional to the slope and the trapezoidal deformation is proportional to the sharpness of curvature, which, in turn, is proportional to the bending moment, it is possible to lay off two curves —one upon the other—so as to predict the location of the first crack that may occur in a concrete beam. The upper curve is of the same order as the slope curve, and the lower curve the same as the moment curve. If the maximum ordinates of these curves are made proportional to the deformation which they represent for a given beam, then the greatest diagonal tension will occur where the joint ordinate is greatest. It is thus possible to determine the location of the first crack in the concrete beam or the place at which the web of an I-beam will buckle for a given length of beam under a given load.

From the equations of beam theory the external shears are equal to the sum of the loads from the center toward the end. By taking the summation of shear areas from the end toward the center of a beam the magnitude of the moment ordinates can be determined. The summation of moment areas, in turn (divided by $E\,I$), gives the tangent of the slope angles along the loaded beam, and the summation of these tangents determines the deflection according to the ordinary theory of flexure. Because the horizontal shears are determined by summing the horizontal fiber stresses, rhombic shear strain is the reaction to the moment stresses. Hence, it is proportional thereto and the shear deflection, therefore, bears a constant ratio to the moment deflection in a given elastic beam for any loading whatever; but this ratio differs in beams of different span lengths because the magnitude of the rotational pure shear distortion depends on the relation between the horizontal section resisting horizontal shear and the vertical cross-section resisting moment stresses. The relative magnitude of the shear deflection in beams of various lengths follows a different proportionate law from the magnitude of moment deflections.

As a steel I-beam has the flange resistance concentrated at the top and bottom, its web stresses are comparable to those of a concrete beam in which the tensile flange resistance is concentrated in the reinforcing steel. A graphical analysis of the rotational strains of the web of the steel I-beam will illustrate the solution of both problems.

In Fig. 13 let $O\,C$ represent the half span with a concentrated load at the center. The moment curves are, therefore, straight lines. The slope curve is of the second degree and the magnitude of the rotational strain represented by $O\,h$ at the end and the ordinates of the curve, $h\,C$, may be taken as represent-

ing the relative magnitude of the pure shear strains. The magnitude of the trapezoidal distortion is proportional to the ordinates measured from $O\,C$ to $O\,B_1$ for the 10-ft. beam; $O\,B_2$ for the 20-ft. beam; and $O\,B_3$ for the 30-ft. beam.

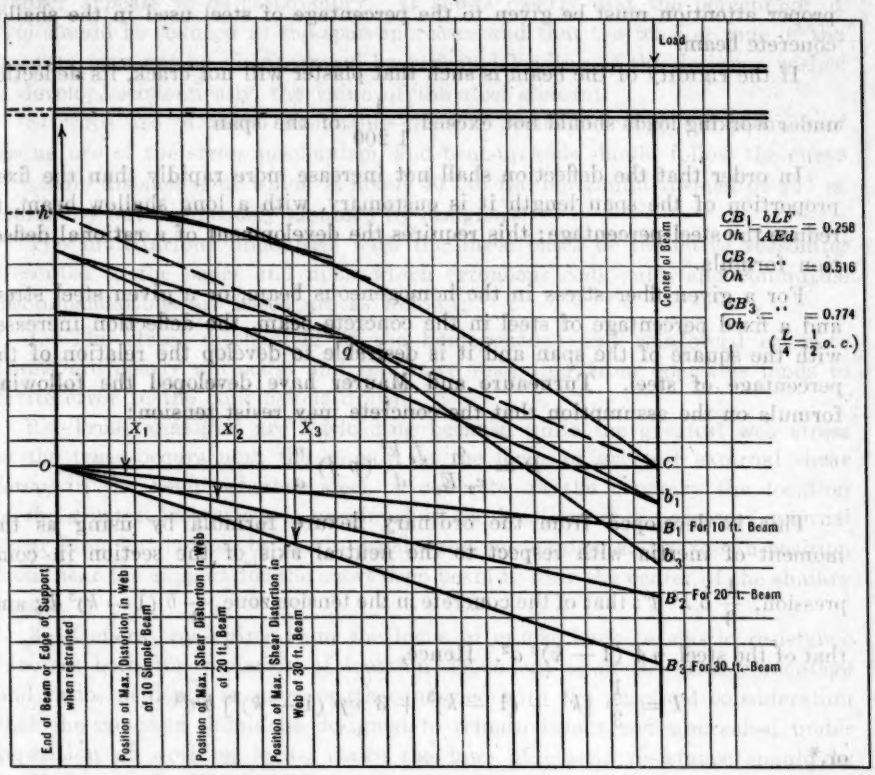
Load
h
q
X_1
X_2
X_3
O
C
b_1
B_1 For 10 ft. Beam
b_3
B_2 For 20 ft. Beam
B_3 For 30 ft. Beam
Center of Beam
$\frac{CB_1}{Oh} = \frac{bLF}{AEd} = 0.258$
$\frac{CB_2}{Oh} = " = 0.516$
$\frac{CB_3}{Oh} = " = 0.774$
$(\frac{L}{4} = \frac{1}{2}$ o. c.)
End of Beam or Edge of Support when restrained
Position of Max. Distortion in Web of 10 Simple Beam
Position of Max. Shear Distortion in Web of 20 ft. Beam
Position of Max. Shear Distortion in Web of 30 ft. Beam

FIG. 13.—ANALYSIS OF ANGULAR SHEAR DISTORTION IN WEB OF 15-INCH, 38-POUND I-BEAM CENTRALLY LOADED, LENGTH SIMPLY SUPPORTED AND RESTRAINED.

The sum of the ordinates of the two rotations is a maximum for the 10-ft. beam at X_1; for the 20-ft. beam at X_2; and for the 30-ft. beam at X_3. Conversely, if the beam is restrained at its ends the moment passes through zero at q and the closing line of the slope curve is $h\,C$. Then $q\,b_1$ and $q\,b_3$ represent the moment curves for the center or suspended-span part, and corresponding curves to the left of q are for the cantilever part. Obviously, the maximum rotational strain in the restrained beam is roughly one-third as great as in the simply supported beam of the same section subjected to the same external shear force. This illustrates the illogical nature of the rules of the Joint Committee on Specifications for Concrete and Reinforced Concrete and building code laws in computing the web stress of beams. Fig. 13 discloses clearly why, as the beam becomes longer and more shallow, the maximum rotational strain moves from the end toward the center in the simple beam and from the quarter-point toward a position between the quarter-point and the support, as the span increases in the restrained beam. It thus becomes

possible to predict with a high degree of certainty where the first crack in a concrete beam will occur, or where the web of the steel beam will commence to buckle. Because the concrete has a smaller limit of rotational strain before cracking than that possessed by the steel before the yield point is reached, proper attention must be given to the percentage of steel used in the shallow concrete beam.

If the rigidity of the beam is such that plaster will not crack, its deflection under working loads should not exceed $\frac{1}{1\,200}$ of the span.

In order that the deflection shall not increase more rapidly than the fixed proportion of the span length it is customary, with a long shallow beam, to reduce the steel percentage; this requires the development of a rational deflection formula.

For a given fiber stress in the homogeneous beam, or a given steel stress and a fixed percentage of steel in the concrete beam, the deflection increases with the square of the span and it is desirable to develop the relation of the percentage of steel. Turneaure and Maurer have developed the following formula on the assumption that the concrete may resist tension:

$$D = \frac{c_1}{c_2 E_s} \frac{f_s k^2}{d} (p\ j) \frac{n}{\alpha}$$

This is developed from the ordinary flexure formula by using as the moment of inertia with respect to the neutral axis of the section in compression, $\frac{1}{3} b k^3 d^3$; that of the concrete in the tension zone, $\frac{1}{3} b (1-k)^3 d^3$; and that of the steel, $nA (1-k)^2 d^2$. Hence,

$$I = \frac{1}{3} [k^3 + (1-k)^3 + 3 n p (1-k)^2] b d^3$$

or,*

$$\alpha = \frac{1}{3} [k^3 + (1-k)^3 + 3 n p (1-k)^2]$$

In explaining this formula Messrs. Turneaure and Maurer note that the concrete may crack, but apparently they are inclined to consider that it resists tension after it has been cracked by tension. However, the fact that horizontal shear in the concrete beam, parallel with the steel, replaces horizontal tensions of the homogeneous beam furnishes the solution of this enigma.

The manner in which horizontal shears may cause or oppose bending in the concrete beam may be illustrated by testing a 12 by 16-in. beam, 14 ft. long, centrally loaded, but resting on symmetrical supports 8 ft. apart, allowing 3 ft. over-hang at each end. With 2% of reinforcement the first crack in the concrete will occur in the top of the beam in the overhanging part where there is no applied moment; hence, it is apparent that the horizontal shear in the concrete, parallel with the steel, actually forms that part of the bending resist-

* "Reinforced Concrete", by Turneaure and Maurer, p. 204, Equation (5).

ance which has been erroneously attributed by theoretical writers to the tensile resistance of the concrete.

With this explanation the moment of inertia developed by Turneaure and Maurer appears to be satisfactory. It shows clearly that the percentage of steel should be reduced as the span increases and that the tension zone of the concrete may not be disintegrated by repeated loading if the designer wishes to develop, economically, the value of the steel element.

Stirrups are of much less value than is commonly supposed because of the nature of the stress mechanism, and bent-up rods should follow the curve of tensile displacement which is about 30° to the horizontal instead of 45° as commonly but erroneously assumed by designers.

The multifarious difficulties with the three bases of inductive reasoning presented in the paper and upon which erroneous code rules and committee recommendations are based are as follows:

1.—The stress mechanism of the homogeneous and reinforced concrete beams are nearly antipodal and inductive reasoning from opposites tends to invite error in the conclusions drawn.

2.—Truss analogies are misleading because while the greatest web stress in the truss occurs next the support at the locus of greatest external shear force, in the beam whether steel or concrete on the contrary the location of the greatest web stress is unrelated to the location of the greatest external shear force but is fixed by the internal stress mechanism and varies in position from near the support for the short deep beam to near the center of the shallow beam.

3.—General reasoning from the locus of cracks back to elastic resistance involves both the variation of location dependent upon the steel percentage and ratios of depth to span length combined with the practical consideration that the concrete should be designed to remain intact and uncracked under repetition of working loads, hence the laws of elastic resistance should be applied to secure this desideratum.

H. E. Eckles,* M. Am. Soc. C. E. (by letter).†—The author's clear presentation of this important subject will be very valuable to structural engineers interested in economical and careful design. It is an unfortunate fact that, from the standpoint of design, web reinforcement is very often one of the weakest features of reinforced concrete structures. This is frequently due to confidence in the effectiveness of vertical stirrups which the author's analyses show clearly is unwarranted. He has stated‡ that the function of vertical stirrups is to delay failure which has already become a fact, and that they do not take any stress before this stage is reached. These are facts which have been established from the time of the earliest reliable tests of reinforced concrete beams, and it has been equally well known that these drawbacks do not apply to the use of bent-up bars. It is rather inconsistent to design a structure for a condition of failure which, when

* Chicago, Ill.

† Received by the Secretary, January 25, 1929.

‡ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 21.

reached, will necessitate its being replaced, particularly when it may equally well be designed so as not to reach this stage, by the use of bent-up bars.

The author has very significantly stated* that bent-up bars may act as web reinforcement before cracks appear in the concrete, and that they act as well to delay failure afterward, in a manner similar to that claimed for vertical stirrups. The statement is also made† that bent-up bars deliver their tension as main reinforcement to the compression side of the beam with the result that questions of diagonal tension are eliminated, and that‡ when diagonal bars are used the unit web shear is one-half that in beams with vertical stirrups. These and other considerations make it highly desirable that careful consideration be given to the use of diagonal bars in preference to vertical stirrups. Bent bars may be made thoroughly effective in a much simpler way than that suggested by the author.

The danger of bent-up bars breaking out at the sides of a beam, or causing a beam to split, is unfortunately exaggerated by laboratory tests of beams to destruction. This danger may be avoided by the use of a short, heavy transverse bar placed on the inside of the bends or hooks. This will keep the bent-up bars away from the sides of the beam; will distribute better the bearing of the bent-up bars on the concrete at the bends; and will tie the concrete together transversely. Short hooks or bends at the ends of this transverse bar may well be adopted in important work, and this avoids two of the principal difficulties in design, namely the use of bends of long radii and the splitting of the concrete at the bends. The exterior bars should be kept straight to avoid the danger of failure shown in Fig. 6.§ Failure, as shown in Fig. 7,§ will be avoided by the use of short, heavy transverse bars with hooked ends, if the beam has uniform bearing.

A third difficulty to be met in the use of bent-up bars is that of anchorage. The author's suggestion (Fig. 9 (*b*))‖ to bend the bars sidewise out of the vertical plane is unnecessary and highly objectionable from the standpoint of bending, transporting, storing, and placing the bars in the structure. When main reinforcement must stop over supports an initial slipping of the bar may often be reduced by using end hooks of short radii and a short transverse bar at the bends, rather than hooks of long radii and free ends. Sufficient anchorage in other cases can readily be secured without resorting to unusual bends.

The importance of good anchorage for the main reinforcement was clearly shown in the earliest important tests of reinforced concrete beams. It was then stated that a greater resistance of the beam would be secured against initial failure by shear if, instead of using vertical web reinforcement, the amount of main reinforcement used at critical shear points was increased. The author's analyses suggest this in a different way. Design that will delay

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, pp. 22 and 26.
† *Loc. cit.*, p. 43.
‡ *Loc. cit.*, p. 32.
§ *Loc. cit.*, p. 37.
‖ *Loc. cit.*, p. 39.

initial failure is much to be preferred to that of merely delaying final failure in a structure that will have to be replaced if the initial stage is reached. Since the concrete takes all the shear when vertical stirrups are used, it is preferable to take the material ordinarily used for vertical stirrups as additional main reinforcement or bent-up bars. This would lower the unit bond stress of the main reinforcement and the neutral axis of the beam at the critical shear point, and thus reduce the tendency for cracks to develop which are caused by initial failure. At the same time, this would give a greater amount of concrete in the section to resist shear, with the result that initial failure would be delayed longer than when vertical stirrups are used in any amount whatever. The author has stated* that the total cross-sectional area of vertical stirrups in one end of a beam will generally equal that of the main reinforcement at the points of both maximum positive and negative moment, from which it will be seen that the amount of steel in the vertical stirrups would increase considerably the main reinforcement if used in this way, and would materially raise the useful limit point of the beam.

The author might have gone further in pointing out the comparatively greater value of diagonal web reinforcement over vertical stirrups. Diagonal bars are much more effective in preventing diagonal cracks in the tension side of the beam for both the positive and negative moment areas, since their direction in both cases is nearly at right angles across such cracks. When the sets of diagonal bars are spaced at a less distance apart than jd, they will be very effective for this purpose; vertical stirrups are of no practical value in preventing diagonal cracks, because of their direction. There is a serious defect in the author's analysis of vertical stirrups by the truss analogy† since this assumption leads to the anomalous situation of having tension in the concrete in the direction of the vertical stirrup in the compression area. This has been emphasized by the conclusion that hooks are needed at the ends of vertical stirrups. An examination of Fig. 2‡ and Fig. 3§ will help to make clear how this tension must be erroneously assumed. If any considerable tension exists in any part of the length of a vertical stirrup the concrete there must be stressed in tension beyond its point of failure, a condition which would develop cracks throughout the length of all vertical stirrups which are assumed to be functioning. This has a special significance when considering the section of a vertical stirrup which is above and to the left of a major crack, as shown in Fig. 2 (a) and Fig. 3 (a). In that area the unit tension in the concrete is that in the vertical stirrup divided by n.

All this raises the question of whether the required tension in the stirrup in the compression area must be assumed to exist only if major diagonal cracks occur there. This, in turn, would require it to be assumed that major cracks exist throughout the length of the beam to the support. This cannot be the case, however, for such cracks exist only at the point where

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 36.
† *Loc. cit.*, p. 24.
‡ *Loc. cit.*, p. 23.
§ *Loc. cit.*, p. 29.

substantial slippage of the main reinforcement has stopped, or where the main reinforcement is greatly overstressed. Tests do not show the existence of numerous cracks such as these before failure has resulted. In the case of continuous beams no substantial slippage can occur where the main reinforcement extends any distance beyond the point of contraflexure, or is properly bent into the compression area of the beam. This renders the use of vertical stirrups entirely futile in such beams. Reducing the main reinforcement over supports merely increases its unit stress and reduces the shearing resistance of the concrete at the most dangerous point, due to increasing the amount of concrete that is broken up by tension in the tension side of the cross-section of the beam. Differences in the moments of inertia of cross-sections at the supports and at the middle of the beam have little effect on the relative moments at these points, since the points of contraflexure are changed but little by these differences. The danger in this region from improper design is well illustrated by Fig. 6.

After all, the concrete must take the shear very largely in all sections of a beam. Tension is developed in diagonal bars to some extent because their direction is at right angles to the diagonal tension cracks in both continuous and simple beams, and, because of this tension in the tension side of the beam in the diagonal bars which is developed by moment, will be of substantial value if properly spaced.

Their effectiveness is apparent by virtue of their being in the direction of rotation about the centroid of compression at a point such as *A*, Fig. 1 (*h*)*. This makes them effective at the beginning of stress in the beam, in contradistinction to the lack of such action in vertical stirrups before initial failure takes place.

Reliable tests show that concrete has a shearing value of about two-thirds that of its compressive strength, and good design requires only that tension in the concrete be taken care of by suitable reinforcement. When the specifications are such as to insure this, the author's suggestion† that higher unit shearing stresses be allowed, may well be adopted. The suggestion to use a higher unit tension stress in the main reinforcement, however, should be examined very carefully. It would reduce the percentage of main reinforcement and hence the area of concrete under compression. In the end it would result in increasing the dead load, in reducing the very effective area for shear on the compression side, and in producing undesirable effects on bond anchorage.

JACOB FELD,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—In analyzing the action of web reinforcement the author has developed his ideas clearly and has checked his theories by comparison with reinforced concrete beams tested to destruction. He might simplify his analysis of the stresses in reinforced concrete by starting with more fundamental ideas than such expressions as "diagonal tension." All strains can be resolved into two types:

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 21.
† *Loc. cit.*, p. 28.
‡ Cons. Engr., New York, N. Y.
§ Received by the Secretary, February 1, 1929.

(*a*) Tension or compression, acting along a line, accompanied by strains of the opposite type acting along the other two axes.

(*b*) Shear which is a strain causing no change in volume and is equivalent to a compression along one axis, a tension along the other axis, and no stress along the third axis.

In other words, all strains must be considered along three axes. Referring to reinforced concrete, tension in the embedded steel rods must be accompanied by compression along the axes at right angles to the rod. Shear stress can be resolved into compression and tension along two axes at right angles to each other. This is the source of the mysterious diagonal tension in concrete beams.

The strength of concrete in compression is greater than in tension. However, all concrete must have some strength in tension, otherwise it could not take shear. The strength of concrete in shear is governed by its strength in tension* and the assumption of a beam made of concrete capable of taking shear but not tension, is misleading. This double assumption is impossible.

In resisting compression, concrete fails more quickly by its inability to take care of the resulting tension strains (Poisson's effect) than it does in resisting compression strains. The writer has never seen an example of failure of concrete due to pure compression strains. Such a failure would only occur if it were capable of resisting considerable tension.

In resisting shear no cracks develop until the tension component of the shear overcomes the ability of concrete to resist it. It is very difficult to understand why the measured values of the tension in web reinforcement do not appear until cracks develop, because, if the concrete takes tension in resisting shear, certainly the embedded steel along such lines must also take it. However, when concrete sets, the embedded steel is compressed, and it is more than likely that whatever tension is developed in the web reinforcement before cracks develop does not compensate for the initial compression in such steel due to the setting of concrete.

In deep beams, which give a large section of concrete toward the resistance of shear stress, failure in test specimens is not in shear if the tension bars are well anchored. Failure in many specimens is due to splitting, as is well shown by Figs. 6,† 7,† and 8,‡ because the compression in the concrete has induced tension stresses at right angles to the axis of the compression and no provision has been made to take such tension. The author's suggestion (Fig. 9(*b*))† that rods be bent into opposite flanges would take care of such tension stresses and would permit higher compression stresses in the concrete before cracks appear in the compression face.

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 30.

† *Loc. cit.*, p. 37.

‡ *Loc. cit.*, p. 39.

(a) Tension or compression, acting along a line, accompanied by strains of the opposite type acting along the other two axes.

(b) Shear, which is a strain causing no change in volume, and is equivalent to a compression along one axis, a tension along the other axis, and no stress along the third axis.

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In resisting shear no cracks develop until the tension component of the shear overcomes the ability of concrete to resist it. It is very difficult to understand why the measured values of the tension in web reinforcement do not appear until cracks develop, because, if the concrete takes tension in resisting shear, certainly the embedded steel along such lines must also take it. However, when concrete sets, the embedded steel is compressed, and it is more than likely that whatever tension is developed in the web reinforcement before cracks develop does not compensate for the initial compression in such steel due to the setting of concrete.

In deep beams, which give a large section of concrete around the neutral axis of shear stress, failure in most specimens is not in shear if the tension bars are well anchored. Failure in many specimens is due to splitting, as is shown by Figs. 6, 7, 7½ and 8.[1] because the compression in the concrete has induced tension stresses at right angles to the axis of the compression and no provision has been made to take such tension. The author's suggestion (Fig. 9(b)) that rods be bent into opposite flanges would take care of such tension stresses and would permit higher compression stresses in the concrete before cracks appear in the compression face.

[1] Proceedings, Am. Soc. C. E., March, 1909, Paper and Discussion, p. 99.
[2] Loc. cit., p. 97.
[3] Loc. cit., p. 98.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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MODERNIZING THE SUBURBAN TRANSIT OF THE METROPOLITAN DISTRICT

Discussion*

By W. B. Yereance, M. Am. Soc. C. E.

W. B. Yereance,† M. Am. Soc. C. E. (by letter).‡—Much has been written on the general subject of New York's "commuter problem" but much more study is needed before an adequate solution of even the transportation side of the problem can be expected. Each article that is written is helpful if only in emphasizing the special points appealing to an investigator or in revealing his viewpoint. That this is no simple problem most of the many who have given it any considerable study have already discovered. It has grown to be highly complex, deeply and inextricably involved with a variety of correlated problems, big in every sense of the word. The writer does not propose to present an invulnerable, complete solution—such attempt would not now be timely; but he wishes to emphasize some factors that are frequently overlooked.

While a negligible number and proportion of the commuter multitude may use other means of transportation, practically all, so far as may now be envisioned, must continue to rely on a railroad service as being best adapted to their needs. The earlier requirements of the commuter were satisfactorily met when the railroads provided daily transportation between the home station and the station in the community wherein his business was located, with reasonable promptness and comfort as measured by the standards of the day. All railroads that lacked terminals on the Island of Manhattan used or provided a ferry service between the rail-head and the Manhattan shore; that necessitated at least one change of conveyance by each passenger to complete this trip. Most of those roads later established multiple ferry routes making possible inbound passenger delivery at more than one point in Manhattan. When such delivery had been effected, the railroad's obligation under the pas-

* Discussion of the paper by Francis Lee Stuart, M. Am. Soc. C. E., continued from December, 1928, *Proceedings*.

† Cons. Engr., and Operator for Public Utilities, South Orange, N. J.

‡ Received by the Secretary, March 6, 1929.

senger's contract was fulfilled. Despite the limitations that any ferry operation suffers, due mainly to weather conditions, this service was highly satisfactory for many years.

The complete commuter service of any large city really presents a compound problem. It involves, in the language of the light and power utilities, both transmission and distribution; in freight parlance, both haul and store-door delivery. For many years an admirable street railway distribution system supplemented the railroad transmission service. Even during horse-car days, the street railway had a diversity of routes, and a relatively good speed was maintained. It provided a real service to practically all sections of the city, usually without the necessity of transferring. Although so many of the former residential districts of Manhattan have now become commercial that the commuters' local destinations are consequently multiplied, the evident fact is that much of the commuter's complaint to-day is due to the unfortunate abandonments, deterioration, and almost complete breakdown of the formerly good local street railway service. In the period of their maximum development the combined ferry and street car services provided a highly satisfactory means of distribution that will be extremely difficult to realize economically again.

With the lapse of time and the practical application of scientific discoveries for man's added comfort and convenience, new ideas of the requirements have been born and new standards created. To-day, a person would indeed be rash to venture to place a limit on the demands of the next half century or to detail such demands. While railroad or transmission service has been much improved, the general criticism is rather that it has by no means kept pace with the more or less reasonable demands of its patrons. Possibly the main grievance of the commuter is the hitherto general lack of recognition by the carriers, a number of which are "trunk line" roads, that the commuter service should be and is, in proper practice as well as in theory, a rapid transit service that cannot be handled satisfactorily by a mere local application of through passenger practice nor by any multiplication of merely local passenger service. In practical operation there is a fairly clean-cut distinction between commuter and local business, although at times and in certain sub-sections of the territory the line of demarcation is far from clear.

The commuter's dream to-day is of immediate, comfortable, and safe transportation at any time, between his dining table and his office desk, without change of conveyance, and at a cost not exceeding that of the local street car fare. Manifestly, this is physically as well as financially impossible. The problem is as to how nearly this dream can be made to come true. Any plan, even when fully completed, will still involve the use of local means of distribution by a considerable number of commuters. To this extent at least any commuter service is and always will be "tied in" with the local service, despite any and all merely theoretical attempts to disassociate them.

Nevertheless, what can and should be done is to develop and, through co-operative effort, inaugurate some plan that will provide rapid, reliable, comfortable, and safe commuter transportation with a through service as direct and convenient as may be economically possible and at minimum cost to all

concerned. There are two irreconcilable factors involved, even with the question of comparative costs eliminated for the moment. Through service cannot be most rapid for any one user unless it is direct and without stops between his terminals. It cannot be most convenient for all users unless it is more or less indirect and with frequent stops. To provide for both present and anticipated requirements, in making a wise adjustment of these two factors, there is needed the vision of a Moses and the judgment of a Solomon.

In any adequate solution offered, full and ample provision must be made of course to serve all outlying sections and to serve them, approximately at least, with equal efficiency. It is to be inferred from the paper* that Staten Island should be grouped with the outlying sections, and offered a commuter rather than a city rapid transit service. While Staten Island (Borough of Richmond) belongs geographically in the State of New Jersey it is both legally and politically as much a part of New York City as the Borough of Manhattan. Should not Richmond, in common with the other boroughs, be served by an extension of some city rapid transit route now or later serving Manhattan, and would it not thereby be better served? If so, there are three routes possible for such extension, as follows: (*a*) Brooklyn and Fort Hamilton; (*b*) Bayonne; and (*c*) Governors Island. The first two are roundabout, long, and well-populated routes that could not provide either rapid or comfortable service with the type of equipment best adapted to a strictly local use. Furthermore, the second would involve an incursion into the territory of another State. The third would necessitate more than five miles of subaqueous tunnel but it would be almost a "bee-line" to St. George and would provide direct rail communication between Manhattan and Governors Island. This Island seems destined to become the site of New York's official airport and will then require rapid and reliable rail service to Manhattan. For many reasons it would seem preferable that the Borough of Richmond, as an integral part of New York City, shall be served by a city rapid transit line which would handle most comfortably a relatively long, two-way traffic. Some day a rational and simple system of fares will be adopted making such operation financially possible. Perhaps the author contemplates merging the commuter routes of the Jersey Central, Pennsylvania, and Lehigh Valley Railroad Companies near Elizabeth, N. J., continuing them thence by way of Howland Hook, North Shore, and (with the Baltimore and Ohio Railroad added), Route (*c*) to a connection with the proposed new Manhattan loop. This is plainly not a mere question of detail; it must be decided before the design of the southern part of the proposed loop could be even outlined, or its successful operation planned.

Whatever plan is adopted must be permanent, as that term is commonly understood. To insure this as nearly as possible, the completed design must be: (*a*) Adapted to present and prospective needs; and (*b*) readily susceptible of expansion, extension, or modification at minimum cost as developments, impossible now to forecast with any degree of certainty, may demand. New York City already has had too many cases of expensive experiments or badly

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1391.

conceived projects requiring extensive revision or total abandonment shortly after their inauguration. Some of these merely illustrate how easily, when dexterously manipulated, the money of others can be squandered. The long-continued and inevitable trend of business northward in Manhattan may be temporarily halted, but evidently it cannot be stopped.

Considering the number and importance of business interests now located as far north as 59th Street it would appear distinctly shortsighted to consider seriously laying the north side of the proposed loop along 57th Street. The proposed improvement should be really permanent and a minimum reliance placed on local distribution services. If 57th Street or 59th Street should, for any controlling reason, be finally decided upon as the northern limit of the proposed loop, it would still be an open question whether the Hudson River crossing there could better be effected by tunnel or by bridge. This question should and must be decided more from the operating than the engineering or financial viewpoint because the value of the monumental enterprise under consideration cannot be impaired by any relatively trifling differences in first cost; only the best plan will prove acceptable.

Such a comprehensive plan as the author outlines should be susceptible of "piecemeal" construction and operation (1) to insure the earliest possible commuter relief that could be expanded as construction progress permits; and (2) to provide an income for application against the carrying charges. At the very best, the work of actual construction will extend for some years after any adequate plan shall have been adopted and financed. If the plan of a complete loop on Manhattan itself is adopted, one of the east-and-west or north-and-south sides, with two connecting river crossings, should be completed, and limited through-routed operation should be inaugurated before construction of the remaining work is begun. It is hardly likely that the citizens or the local authorities would tolerate such a widespread interference with Borough life as would necessarily result from an attempt to push work coincidently on all sections of the entire plan, were the required labor available without utter demoralization of the labor market, injurious effect on other necessary projects, and a considerably increased labor cost. In any plan of operation, temporary or permanent, U-shaped routing should be avoided, except where topographic conditions necessitate it. For instance, routing over the Central Railroad of New Jersey and the New York, New Haven, and Hartford Railroad would be better than over the Central Railroad of New Jersey and the Lackawanna route (also *via* Manhattan) as passengers between the latter localities can be handled more quickly and cheaply directly between Elizabeth, Newark, and Paterson, than through the congested center.

The cost of the plan outlined by the author is given as from $1 250 000 000 to $1 500 000 000*. Probably a better guess would be $2 500 000 000 to $3 000 000 000 before a complete plan is physically realized and ready for operation. Can the mind of man appreciate this colossal expenditure? This amount is more than one-tenth the Interstate Commerce Commission's valuation of all the railroads of this country, equipped and ready for business as going concerns. It represents the value of more than 25 000 miles of operating rail-

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1393.

road, or more than the combined mileages of the great New York Central and Chicago, Milwaukee, St. Paul and Pacific Systems with all their subsidiaries.

This is surely the time and the place to "Stop, Look, Listen". Relatively little if any of this cost should, or presumably would, be met by special assessment on private property which, in this territory, is now carrying about all the tax load it can stand. Local authorities can be depended upon to further increase assessed valuations in full recognition of any possible advantages accruing. The assumption of a "3½% or lower"* interest rate on this stupendous amount seems rather optimistic and hardly warranted by the present or prospective conditions of the money market. The best authorities seem to agree that the days of cheap money are past. Quotations of the conservative tax-exempt municipal or State bonds clearly indicate this. The revenues are properly assumed derived mostly from increases in fares to be paid by the users; but this increase would be roughly between 50 and 200% and, if experience is of any value, will result in reducing materially the number of probable users, and hence the revenue to be secured.

Any adequate plan adopted for commuter relief, or passenger service only, would involve an enormous investment in an extensive plant. Even with all through operation, that is, no looping or turnbacks on Manhattan, the denser, central section, when fully utilized, will be a very "hot" piece of track during the so-called rush hours; but it will be little used during mid-day and evening, and practically idle at night. The load factor, consequently, must be low, probably less than 40 per cent. The greatest industries which are relatively much smaller than this project, could not survive under a maximum of 40% of capacity production or operation except as long as their accumulated reserves might last. How then could a railroad project reasonably hope to prove economically successful under a similar limitation, laboring under both legislative and economic restrictions?

It is no secret that the continuing success of prosperous railroads is attributable almost entirely to the remarkable improvements effected in operating efficiency. It is certainly not due to any adequate rates allowed or any reduction in material costs, labor rates, or tax imposts. There is every economic and financial reason why such a plan as the author describes should provide for both freight and passenger service. Freight of course should be handled in off-peak hours and not conflict with the passenger service.

The outstanding and incontrovertible fact in the paper is the author's statement† that "individual effort would be ineffective and could not succeed". Co-operation must be secured between all National, State, and local governmental authorities concerned, all railroads in the territory, bankers, and the investing public and potential patrons. This simply means that practically all who are in any way affected must be convinced of the advantages that will accrue to each group. Assuming this co-operation, the Federal Government, which is concerned with means of National defense as well as improvement of the economic life of its citizens, would facilitate (through the War Depart-

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1393.

† *Loc. cit.*, p. 1392.

ment) authority for proper river crossings. The Interstate Commerce Commission would authorize a remunerative system of rates for the interstate services rendered. The States of New York and New Jersey would provide necessary authorization for procedure and through their commissions would approve adequate intra-state service rates. The City of New York, having still more to gain, has still more to do; that is, hold in check any obstructive influence, authorize the exercise of necessary functions, and accomplish an effective street traffic control. Instead of involving further large expenditures, this last requirement evidently can be met:

(1) By abolishing day-time parking on any main highway or in any central district streets;

(2) By eliminating the "cruising" of taxicabs; and

(3) By reducing day-time trucking to a minimum.

It now requires 37 to 39 min. to travel on the Broadway line from 8th to 39th Streets. When the necessary street traffic relief has been obtained and the street railway companies are again accorded the opportunity to exercise their full franchise rights in serving the public, this condition will be corrected.

The street railways, with ample capacity for several times the number of their present discouraged patrons, will then resume their proper function as distributing agencies and will again demonstrate that they offer the best means to-day for short-haul, local, mass transportation. When any better is developed it will inevitably be adopted, but nothing of the kind is as yet in sight.

Most of the railroads naturally will be extremely cautious in their commitments because participation in any such enterprise will entail early electrification of their lines from practically the outer limits of their commuting territory to the point of connection with the delivery scheme. Furthermore, until impending mergers are more clearly developed (not, however, to the ultimate two-system plan), other and joint improvements cannot be intelligently undertaken.

The author gives no hint as to his idea of the ownership, either immediate or ultimate, of the extensive plant required in the plan outlined. He does indicate that rent is to be paid by the new company for its use, but not to whom that rent would be paid. Any idea that the entire cost could be met by a bond issue without definite and substantial guaranty is clearly untenable. Without a fairly "thick cushion" behind them such bonds could be marketed, if at all, only as speculative securities, and as such they would entail a high interest rate. As a matter of fact, there is no reason why such a plant, including roadway, structures, and equipment, should be considered or treated otherwise than are the similar terminal plants of the New York Central, Pennsylvania, or any other railroad. This enterprise should be on identically the same basis of ownership and supervision, being brought into existence and sponsored by all who are interested in serving the traveling public for a remuneration.

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1392.

The suggestion is, therefore, in order that a corporation be organized to construct, own, maintain, and operate this property. It should have all the powers of any other railroad and should serve as a terminal road or switching or transfer road with expanded functions. Stock to a total of the minimum physical value of the completed roadway, including land occupied by necessary structures, should be issued and taken at par by all railroads in the territory on some equitable basis of apportionment. Long-term bonds, 40 or 50 years, for example, should be sold to the public in an amount representing the remainder of the cost of the completed plant, including structures, equipment, except rolling stock, and all overhead charges. These bonds, guaranteed by the carriers, should be amortized through a sinking fund provided from revenues and additional securities, on the same basis, and should be issued only for extensions and betterments.

Passenger revenues should be on the basis of passengers or passenger-miles, rather than on car-miles, in order to eliminate any tendency to overcrowd the cars, with a minimum loading guaranteed, to protect the terminal road's revenue. Freight revenue would be based on the ton-mile and would include terminal charges on the several classes for handling, but not for storage. These would be net rates determined by deducting from the gross rate, for each type of service or class of freight, a proper remuneration to the individual roads for the use of cars and crews while on the terminal company's line.

Early agreement is desirable as to what agency could best directly formulate such a plan, develop its details, and complete the construction required. Successful early accomplishment could more reasonably be anticipated if all those whose co-operation is required, especially those more immediately concerned, were active participants as members of a commission authorized by each State and recognized by the Federal Government. Any such commission, composed of only politicians or only railroad men, would make very slow progress.

There are in existence to-day, two bi-State agencies—the Bridge and Tunnel Commission and the Port Authority. Each is composed entirely of political appointees, and each is proceeding under the authority of both New York State and New Jersey. The former has built one highway bridge and one highway tunnel; the latter has made an elaborate report on freight conditions at the port and has presented a plan for their relief. It has built two highway bridges and has a third under construction. In this entirely arbitrary division of province there is too much evident duplication and, what is more serious, opportunity for conflict and expensive competition. The obvious suggestions are that the Bridge and Tunnel Commission be given jurisdiction over the two bridges already constructed by the Port Authority and over the one now under construction on its completion. Furthermore, this Commission should have sole jurisdiction over all interstate river crossings for highway use.

The Port Authority should be re-organized so as to include representatives from each State, the City of New York, each railroad system, and a resident member of the American Bankers' Association. This body, with powers ex-

panded to cover both passenger and freight services, should have similar jurisdiction over all interstate river crossings for railroad use as well as similar intrastate crossings at the Port of New York. As a special assignment, with necessary extended powers, the Port Authority would also be given charge of the design and construction of an adequate plan to combine relief for commuters with the salient features of its predecessor's plan for freight relief.

All costs of design and construction to points of junction with the individual roads, or combination of roads, would be met, of course, by the new terminal corporation.

MEMOIRS OF DECEASED MEMBERS

HEZEKIAH BISSELL, M. Am. Soc. C. E.*

DIED JUNE 23, 1928.

Hezekiah Bissell was born at East Windsor (Scantic), Conn., on February 7, 1835, the son of John Bissell and Elizabeth (Thompson) Bissell. He was a direct descendant of John Bissell who settled in Windsor, Conn., in 1639, and a year later removed to the east side of the Connecticut River.

Mr. Bissell attended the schools of East Windsor and Williston Academy at Easthampton, Mass., where he prepared for college. He entered Sheffield School at Yale University in 1857, but three months before his graduation in 1861 he left school and enlisted at Hartford, Conn., in Company A of the First Regiment, Connecticut Volunteers. He was one of the first to answer the call for the three months' volunteer service, enlisting on April 18, 1861, serving his term of ninety days, and being mustered out on July 31, 1861. On August 23, 1862, he re-enlisted in Company G, 25th Connecticut Volunteers, and was chosen First Lieutenant of that Company, which was largely made up of volunteers from his home town of East Windsor and the neighboring towns of South Windsor and Glastonbury, Conn.

This regiment served with General Banks in the Louisiana Campaign, and was engaged in the siege of Port Hudson and the Battle of Bayou Teche. Mr. Bissell's second service being a nine months' enlistment, he served his full time, and was mustered out on August 26, 1863, so that his complete service was for nearly sixteen months.

With the other students of Yale who left college to enter the Army in 1861, he was given the degree of Bachelor of Philosophy in 1881.

As an example of Mr. Bissell's oneness of purpose the following is taken from his personal memoirs: "When a pupil in Williston he was asked by Mr. J. M. Stoughton, an old friend of his father, what he intended to do, and his reply was, 'I am going to help build the Pacific Railroad; then go to South America to build more railroads; then for my real life work be Engineer of some first-class railroad in the United States; later, I will invent something to give me all the money I want to enjoy life'." And in his personal memoirs written after his retirement from all business, Mr. Bissell adds this statement, "I have done all but the last of these."

Soon after his discharge from the Army Mr. Bissell went to New York, N. Y., and at the office of the Union Pacific Railroad Company in that city, met the late Thomas C. Durant, F. Am. Soc. C. E., President, and Peter A. Dey, Chief Engineer, of the proposed railroad. As a result of this interview he was appointed Second Assistant Engineer by Gen. John A. Dix and, on April 1, 1864, reported at that Company's office at Omaha, Nebr. From this date until its completion on May 10, 1869, he was engaged on some part of the great undertaking.

* Memoir prepared by Lester W. Tucker, M. Am. Soc. C. E., and Walter H. Norris, Assoc. M. Am. Soc. C. E.

His positions on this work were that of Second Assistant Engineer, First Assistant Engineer, Division Engineer, and Resident Engineer, but in the latter part of his service much of his time was spent in special assignments on various important undertakings. As Locating Engineer, Mr. Bissell located and built much of the original line from Laramie, Wyo., to west of Green River, and he was one of the party who ran the preliminary trial lines from the south end of Great Salt Lake westerly into what was then called the Great American Desert, and, with this party, endured great hardship. He was the only Engineer on the work who stayed with the undertaking from the beginning of the lines to its completion, a period of more than five and one-half years. In his own words, he had "staked out the first culvert at Omaha and was present when the gold spike was driven at Promontory Point."

In 1901, Mr. Bissell, while on a trip to the West, stopped in Omaha and met Mr. Burt, then President of the Union Pacific Railroad Company. Mr. Burt invited him to ride over the road with which he had been so intimately connected thirty-five years before, furnishing him a car and the Division Engineer as escort, and arranging for the entire trip to be made by daylight.

Again, in 1919, on the occasion of the Fiftieth Anniversary of the driving of the gold spike, Mr. Kennedy and Mr. Bissell were guests of the Railroad Company, and were the only representatives of the original engineering officials.

Mr. Bissell's work on the Union Pacific Railroad closed in June, 1869, and he returned East, but had been there only a short time when he was asked by Dr. Durant, the former President of the Union Pacific Lines, to undertake the location of the Adirondack Railroad then proposed to be run from Ogdensburg, *via* Canton, to Saratoga, N. Y. Surveys on this line were started late in August, 1869, and from that time until the fall of 1870 the work of surveying and grading was carried forward; but during 1871, money being scarce, little was done, and the work on this line was closed.

Early in February, 1872, he received an offer to go to South America on the roads then being built and projected by Mr. Henry Meiggs for the Peruvian Government. He sailed from New York on May 15, 1872, and after a very stormy passage to Colon (then called Aspinwall), and a slow trip down the West Coast, he finally landed at Mollendo, the Port of Arequipa, Peru.

His work was that of locating and building the railroad from Juliaca, northerly up the Pucara River for a distance of about 100 miles, this being on what was projected as the main line from Puno, a port on Lake Titicaca, to Cuzco, the ancient Inca Capital of Peru. This section, being northerly of Lake Titicaca, was located on the backbone of the Andes Mountains, and was from 12 000 to 14 000 ft. in elevation for its entire length.

The 100 miles were located and graded complete and ready for track and Mr. Bissell was ordered to establish his camps and get ready for the location and grading of the remainder of the line into Cuzco when he received sudden orders to stop all work and leave the country at once. A revolution of great intensity was under way in the northern districts of Peru, and all Americans had to flee.

Mr. Bissell was engaged for two years on this work during the entire period at these high altitudes with few, if any, white men, and at a time when it was necessary to live as the natives lived; he always thought that his thin hair and beard were one of the results of his mode of living during these two years.

Returning to his home from this work early in 1875, he received an offer to go with the Cleveland, Cincinnati, Chicago and St. Louis Railway Company and for the next three years was with that road in various engineering capacities, revising locations, examining bridges, and constructing cut-offs from Cincinnati, Ohio, to the north and west.

On February 1, 1878, Mr. Bissell was appointed Master of Maintenance of Way of the Eastern Railroad Company extending from Boston, Mass., to Portland, Me., with headquarters at Salem, Mass. The Eastern then had 270 miles of road. In 1884, it was taken over by the Boston and Maine Railroad Company and Mr. Bissell was made Chief Engineer of the Boston and Maine Lines, with headquarters at Boston. In 1886, the Boston and Lowell Lines were taken over by the Boston and Maine, and, in 1900, the Fitchburg Lines came under the same management, so that Mr. Bissell became the Chief Engineer of one of the largest railroads in the East with a mileage of 2 300.

While acting as Chief Engineer of the Boston and Maine System, Mr. Bissell had charge of the rebuilding of much of the roadbed, many of the bridges, and the greater part of the railroad stations on that line. He relocated many miles of the first road, double-tracked much of the original Eastern and Boston and Maine Lines, and reduced grades and curves, eliminated grade crossings, and built many new yards, terminals, wharves, and docks.

On June 30, 1909, Mr. Bissell resigned his position as Chief Engineer after having served thirty-one years and, in his announcement, President Tuttle retired him with the title of Consulting Engineer and paid him an exceptional compliment of appreciation for his long and faithful service. The following year he went to California where he made his home in Altadena.

Mr. Bissell was a man of rare intelligence, great integrity, and oneness of purpose. He held the respect and loyalty of all those with whom he came in contact whether they were the men of his own organization or those of the contractors and builders with whom he was so closely associated. His directness of purpose, absolute honesty, and ability to see the other fellow's point of view made engineers and contractors alike respect him and his decisions. He was especially kind and helpful to the young engineers of his organization.

Mr. Bissell was twice married; his first wife was Alice Hughes, of St. Thomas, Ont., Canada, to whom he was married in 1875 and by whom he had three sons, all living. His second wife, Segrid Johansen, to whom he was married in 1910, died several years later.

Mr. Bissell was a member of the Yale Engineering Association, Boston Society of Civil Engineers, Southern California Association of Engineers, Engineers' and Architects' Association, and the Yale Alumni Association. During his long residence in Boston he was a member of many of the leading social and literary clubs of that city, but withdrew from most of these when

he went to California. In his early manhood he was a member of the Congregational Church, but after his marriage he united with the Protestant Episcopal Church.

Mr. Bissell was elected a Member of the American Society of Civil Engineers on September 15, 1869, and at the time of his death was the third oldest member of the Society, being exceeded only by the late Cady Staley, M. Am. Soc. C. E., who was elected on March 17, 1869, and by Clemens Herschel, Past-President and Hon. M. Am. Soc. C. E., who was elected on September 21, 1869. Mr. Bissell served as a Director of the Society from 1905 to 1907.

JAMES HOPKINS DOUSMAN, M. Am. Soc. C. E.*

DIED JANUARY 5, 1925.

James Hopkins Dousman was born at Milwaukee, Wis., on September 24, 1863. After completing the courses in the public schools at Milwaukee, he entered the University of Wisconsin, at Madison, Wis., from which he was graduated with the Class of 1884. Even in early life he displayed an energy which was always one of his most prominent attributes. By utilizing all his vacations and other spare time, he managed to acquire the equivalent of one and one-half years of actual experience. During this period he worked for the County Engineering Department at Milwaukee, the Chicago, Milwaukee, and St. Paul Railroad Company, and for the late C. B. Davis, M. Am. Soc. C. E., a Consulting Engineer, at Chicago, Ill.

After graduation, his first position was that of Transitman with the United States Engineer Corps; subsequently, for three years he acted as Deputy County Surveyor of Edwards County, Texas; and then from 1888 to 1892 as Engineer for the Land and Station Department of the Galveston, Harrisburg, and San Antonio Railroad Company.

In 1893, Mr. Dousman organized the firm of Dousman and Sheldon, Engineers, of Milwaukee, which firm was succeeded in 1895 by that of Dousman and Austin, Consulting Engineers, also of Milwaukee. During this period of private practice, Mr. Dousman was connected with a number of important projects, including work for the Chicago, Milwaukee and St. Paul Railroad Company and water-works and sewage design for the Cities of South Milwaukee, and Delevan, Wis., and others. Also, during this period he invented an automatic pressure regulating valve for stand-pipes and other hydraulic work. These valves were used in a number of cities, and a special installation was made for the Cataract Construction Company, at Niagara Falls, N. Y. Mr. Dousman also invented the "Dousman hangers" for carrying falsework on steel construction when concrete or terra cotta is used for fireproofing. In 1898, he became associated with the Milwaukee Engineering Company and held the office of Secretary of that Company until 1902. From 1902 until 1906, he was again engaged in private practice in Wisconsin, specializing in water-works and reinforced concrete design.

* Memoir prepared by N. T. Veatch, Jr. and Bert C. Wells, Members Am. Soc. C. E.

In 1906, he took charge as City Engineer of an important water-works improvement at Trinidad, Colo., which work he finished in a highly creditable manner. He then went to Kansas City, Mo., as Representative of the American Well Works of Aurora, Ill., in Kansas, Oklahoma, Western Missouri, and Southern Nebraska, and continued this connection until his death.

There were few, if any, who could equal Mr. Dousman in the energetic way in which he not only sought and obtained contracts, but also in the way these contracts were completed. In addition to serving his firm well, he rendered a great service to the purchaser and the public in expert and trustworthy counsel and advice. He was successful, and universally liked and respected, and his passing is deeply mourned by all who knew him. He not only had the respect and friendship of those to whom he was selling, but also that of his competitors. Until his last sickness, Mr. Dousman's entire life as an engineer had been spent in an active and earnest effort to discharge each task undertaken with credit, and in this he succeeded to an unusual degree. During his experience as Machinery Salesman a competitor once said: "I have never heard any one, anywhere, question Jim Dousman's honesty, and for energy he has anybody beaten I have ever seen."

Mr. Dousman died at his home in Kansas City on January 5, 1925. He is survived by his widow, Ina M. Dousman, two sons, Donald J., of Kansas City, and Albert H., of Long Island, New York.

Mr. Dousman was elected a Member of the American Society of Civil Engineers on April 3, 1922.

HAROLD WELLINGTON HORNE, M. Am. Soc. C. E.*

DIED MAY 23, 1928.

Harold Wellington Horne, the son of George and Mary (Barnard) Horne, was born at Belmont, Mass., on October 12, 1873. He was graduated from Harvard University with the degree of Bachelor of Arts in 1894 and from the Lawrence Scientific School, Magna Cum Laude, with the degree of Bachelor of Science in Civil Engineering in 1896.

Mr. Horne's first employment was as Rodman with the Metropolitan Water Board of Massachusetts, in September, 1896. Later, he was advanced to the position of Transitman and remained on this work until March, 1906, having been engaged on various structures for the distribution of water to Boston, Mass., and surrounding municipalities.

From March, 1906, to January, 1912, he served as an Assistant Engineer with the Board of Water Supply of the City of New York, first, on location surveys for the Catskill Aqueduct within the Hudson River Division and, later, in charge of the construction of a section of the Aqueduct which included the Bull Hill Tunnel and the Foundry Brook Pipe Siphon.

In January, 1912, Mr. Horne became associated with the Board of Water Commissioners of Hartford, Conn., in the capacity of Division Engineer on the work then being undertaken for an additional water supply. At first, he

* Memoir prepared by Samuel D. Dodge, Esq., Suffern, N. Y.

was occupied with the extensive topographical and subsoil surveys for reservoir and dam locations and title search of large areas of land takings. After this he took charge of the construction of the various structures comprised in the Nepaug Reservoir, including the large earth dam at Philips Brook, several miles of relocated highway, tunnel, and grade conduit, together with 8 miles of 42-in. pipe and the main dam, which was a high concrete structure. Later, he undertook the construction of the long earth dam at New Hartford, Conn., for the compensating reservoir. This work was concluded in March, 1918.

From March, 1918, to December, 1921, he was Assistant Division Engineer with the Miami Conservancy District of Ohio in charge of the construction of the Englewood Dam for flood control of the Miami River.

There followed an interval of about five years devoted to private affairs and certain business enterprises in which engineering played no large part. Mr. Horne, however, became convinced that permanent retirement was not wise. His education, his experience, and his native talent all combined to make engineering the most useful as well as the most congenial field for his effort. Consequently, he resumed his professional work.

In October, 1926, he was called to the Metropolitan District Water Supply Commission of Massachusetts, as Assistant District Engineer. On June 1, 1927, he was promoted to the position of District Engineer in which capacity he continued until the time of his death. He was given general charge of the preliminary surveys and sub-surface explorations for the aqueduct tunnel from the Swift River Reservoir to Wachusett Reservoir and, subsequently, of the construction of a part of this tunnel. He lived to complete only the shafts.

To his friends and associates is left the memory of a kindly spirit, a wide culture, especially in the arts of music and literature, and a soundly educated mind. They recall, moreover, the singular fidelity and painstaking devotion to whatever task was in hand, tedious and uninspiring though it sometimes was, which were the true expression of his ingenuous and honorable nature.

Mr. Horne was married in 1915, to Anna Garfield Davis who, with two children, Richard and Mary Barnard, survives him.

He was a member of the New England Waterworks Association and of the Harvard Club of New York.

Mr. Horne was elected an Associate Member of the American Society of Civil Engineers on October 2, 1907, and a Member on December 3, 1913.

JULIUS MERRIAM HOWELLS, M. Am. Soc. C. E.*

DIED APRIL 22, 1927.

Julius Merriam Howells was born in Bowling Green, Ohio, on March 26, 1859, the son of Joseph and Ruth Merriam Howells. His childhood was spent on a farm in what he termed, "the Black Swamp", near Toledo, Ohio. His

* Memoir prepared by Philip Schuyler, M. Am. Soc. C. E., and Hamilton R. Howells, Assoc. M. Am. Soc. C. E.

father, Joseph Howells, was progressive in thought, adopting the new school of medicine in his practice as a physician, and the new teachings of the Swedenborgian Church in his religion in place of the old orthodox beliefs. Remuneration from his medical practise among the farmers being inadequate for his large family, he moved to Richmond, Ind., where he entered the drug business. Joseph Howells' idea of justice was so unique that upon arriving at a degree of success, his conscience forbade him to enjoy such an easy life, so he again took up his more moderate practice as a physician. This quality of mind was inherited to some extent by his son, Julius, who loved his profession as an engineer, but had little regard for the money it brought him.

Julius Merriam Howells received his education at Earlham College, Indiana, and, later, from the late James E. Mills, M. Am. Soc. C. E., Geologist, of California. Like many others of the older engineers, Mr. Howells had his first practical experience in railroad location and construction for the Monarch Pass, Gunnison and Dolores Railroad, in Colorado (never built), and the Denver and Rio Grande Railroad in Marshall Pass and on the Rio Grande River, in Colorado. During 1881 and 1882 he was with the Maintenance of Way Department of the Pennsylvania Railroad Company, at Pittsburgh, Pa. From 1882 to 1884, he made studies of the tertiary history of the Sierra Nevadas, and mineral surveys under Mr. Mills, with Agassiz and Shaw, Geologists, his headquarters being at Quincy, Calif. From 1884 to 1888, he was Assistant City Engineer, and then City Engineer, of Richmond, Ind., at which time he became a Member of the Western Society of Engineers. From 1888 to 1892 he practised as Consulting Engineer in Chicago, Ill. His practice included work for the Municipal Investment Company, a Chicago firm, and he also served as Chief Engineer for Fostoria, Ohio, where he designed and built the entire system of water-works.

From 1892 to 1894, Mr. Howells was Chief Engineer of the Santa Fé, N. Mex., Water Company, at which time he did his first work in hydraulic-fill dam construction. He built the Santa Fé Canyon Dam, where he experimented with the hydraulic-fill process. He then spent a year as Chief Engineer of the Tyler, Tex., Water Company, where he designed and constructed the Tyler Dam, the first hydraulic-fill dam ever built.* He also represented the Municipal Investment Company, of Chicago, as Engineer-Inspector of properties scattered over half the United States. In 1895, he became Chief Engineer of the Santa Fé Gas and Electric Company, for the Municipal Investment Company. From 1895 to 1898, he was Chief Engineer of the San Diego Flume Company, and built the La Mesa Dam. In 1898, he designed the Pine Valley Dam, which was never completed. From 1899 to 1901 he was Consulting Engineer for the San Joaquin Electric Company, for which Company he built the Crane Valley Dam (hydraulic-fill and rock-fill).

In 1901 Mr. Howells became associated with the late James Dix Schuyler, M. Am. Soc. C. E., of Los Angeles, Calif., which association and friendship continued until the death of the latter. In 1903, he built the Lake Francis Dam (hydraulic-fill), for the Bay Counties Power Company, now the Pacific

* "Reservoirs for Irrigation, Water Power, and Domestic Water Supply," by James Dix Schuyler, M. Am. Soc. C. E.

Gas and Electric Company. It was in 1901 that Mr. Howells conceived the Big Meadows and Feather River hydro-electric development; and from 1903 to 1905 he worked at interesting capital, promoting the Feather River project, obtaining early water rights, and commencing planning and construction.

From 1905 to 1909 he was employed by the Kobe Syndicate in the development of the Anglo-Japanese hydro-electric project, in Japan, following which he spent nearly a year in Switzerland. During 1910 to 1912 he was in Porto Rico as Chief Engineer of an irrigation system near Barceloneta, for which Mr. Schuyler was Consulting Engineer.

From 1912 to 1927 Mr. Howells carried on a general private consulting engineering practice in San Francisco, Calif., his clientele including the East Bay Water Company; the Western Canal Company; the Ocheco Irrigation District; the Reclamation District No. 784; the Montague and Granada Irrigation Districts; the Anglo and London Paris Bank; the Anglo California Trust Company; the James D. Phelan Estate; the Dodge Land Company; Girvin and Miller (bonds); Stephenson and Company (bonds); the Farm Land Investment Company of Fleishackers; the James Irvine Water Development, etc.

For this same period—1912 to 1927—he was Consulting Engineer for the Great Western Power Company for which he designed and built the Big Meadows Dam (hydraulic-fill) in 1913, and the Butte Valley Dam in 1923. In 1926, he engaged in the enlargement of the Big Meadows Dam.

Mr. Howells had an unusual imagination and faculty of accurately visualizing structures in their completed state, long before their construction. His natural ability to estimate quantities and costs of engineering works amounted to genius. His associates would laboriously prepare an estimate only to find that they verified his simple estimate given weeks before without any apparent effort. He had a natural knowledge of proportion and logic, viewing all problems in their entirety without being confused by details. He was an idealist in his profession as he was in his private life. He had a rigid code of business ethics, and a very fair but strict idea of justice.

He readily grasped the economic and political aspects of problems and, due to this trait, found himself useful as an engineering diplomat and arbitrator in controversies among reclamation districts in co-operating with the State plan of reclamation. Loyalty to his friends and associates was probably his strongest trait. He took great pleasure in his work and keenly enjoyed the engineering profession. He was an exceedingly hard worker and preferred to work rather than to pursue the usual diversions of business men.

One of Mr. Howells striking characteristics was his unusual engineering economies. He constantly indicated to his associates where savings of money and effort could be made by using original and unique processes—many so simple that others overlooked them. To illustrate this, when the consolidation of an earth dam was being contemplated and intricate methods of compacting were being discussed, Mr. Howells accomplished a superior and cheap consolidation of the fill by arranging with a near-by goat farm to borrow a band of goats to drive to and fro across the fill. This ingenuity was also exhibited in his rather inventive turn of mind.

He was a member of the Institution of Civil Engineers of Great Britain and Chairman of the Economic Committee of the Immigration Section of the Commonwealth Club of California.

In his private life Mr. Howells was also the idealist. He stood for the very highest code of morals and might have given the impression of being intolerant of violators of his strict code. He was devoted to his home and family. He was a great lover of music and provided a musical training for each of his children. He was extremely interested in plant and animal life as well as in the geologic structure of the earth. He possessed a great familiarity with these subjects and made frequent allusions to them when in the field on business.

His other interests were: The Commonwealth Club and the League of Nations Non-Partisan Association. He stood for the Republican Party in politics. He was a member of the Swedenborgian Church, but during his last few years he was affiliated with the Unitarian Church, being interested in its educational lecture form of sermons.

He is survived by his widow, two sons, Hamilton R. Howells, Assoc. M. Am. Soc. C. E., and Merriam J. Howells, and two daughters, Mrs. George F. Buckingham and Mrs. Southall R. Pfund.

Mr. Howells was elected a Member of the American Society of Civil Engineers on June 4, 1902.

RICHARD LEWIS HUMPHREY, M. Am. Soc. C. E.*

DIED NOVEMBER 2, 1928.

Richard Lewis Humphrey was born in Marblehead, Mass., on October 19, 1869, the son of Richard Hales and Caroline (Curtis) Humphrey and a descendant of Benjamin Hales Humphreys, who came from Wales early in 1600 and settled in Marblehead. His father, who dropped the final "s" from his name, was a manufacturer of shoes in Marblehead and, later, in Philadelphia, Pa., and lived to be nearly ninety years of age. The son was graduated from the Central High School of Philadelphia with the degree of Bachelor of Arts in 1888 and Master of Arts in 1893, and from the University of Pennsylvania in 1891. He later received the degree of Civil Engineer from this institution, and subsequently took a special course in chemistry there.

Mr. Humphrey began his professional career in 1890, during the summer vacation of his Senior year, as Assistant and Topographer to the Division Engineer, on the Cumberland Gap Extension of the Louisville and Nashville Railroad Company. After his graduation, in 1891, he accepted a position as Draftsman in the service of the Philadelphia and Reading Railway Company, resigning in 1892 to become Draftsman and, later, Engineer in the Bureau of Surveys, Philadelphia. He was assigned, successively, to work on plans for the Cresheim Arch of the Wissahickon Creek high-level sewer, main and branch sewers, Front Street plate girder bridge (100-ft. span), and the drainage of the meadow lands in South Philadelphia. He was Designer

* Memoir prepared by a Committee consisting of George S. Webster, Past-President, Am. Soc. C. E., and Samuel T. Wagner and S. M. Swaab, Members, Am. Soc. C. E.

and Engineer in charge of the Philadelphia Municipal Testing Laboratory. This laboratory grew in importance and in it much valuable research work was carried on, particularly in the development of the use of cement, concrete, and reinforced concrete as materials of construction, and Mr. Humphrey was considered a pioneer in work of this character.

In 1899, he became Engineer and General Manager of the Buckhorn Portland Cement Company, and during the ensuing four years he designed, built, and operated this Company's Portland cement plant at Manheim, W. Va. In 1903 he became a Consulting Engineer and Expert, serving the Pennsylvania Railroad Company, and other large corporations. He was Engineer in charge of the construction of the reinforced concrete Exhibition Building and of the assembling and erection of the collective American Portland cement exhibit and model testing laboratory of the American Portland Cement Manufacturers, which exhibit received the Grand Prize at the Louisiana Purchase Exposition, at St. Louis, Mo., in 1904. He served as Cement Expert of the United States Reclamation Service and Cement and Concrete Expert of the United States Geological Survey, and was Engineer in charge of the Government Structural Materials Testing Laboratories in St. Louis, Pittsburgh and Northampton, Pa., and Atlantic City, N. J., from 1905 to 1910, which laboratories he planned. He was Chairman of the Committee appointed by the Government to investigate the effects of alkali on concrete and reinforced concrete used in the construction of United States Reclamation Projects.

Mr. Humphrey also made, in 1906, an investigation for the Government of the effect of the San Francisco earthquake and fire on structures and structural materials, the results of which were published as a Government *Bulletin*. He established and designed the laboratory at Northampton, Pa., for the testing of the Portland cement used in the construction of the Panama Canal and was in charge of it until 1910. From 1910 until 1920 he was Consulting Engineer to the United States Bureau of Mines. During a part of 1910 he was also Consulting Engineer to the United States Bureau of Standards. He was the Expert retained in connection with the revision of the New York City Building Code in 1908 to 1909.

During the World War Mr. Humphrey was appointed (on October 7, 1917, as a "dollar a year" man) an Expert on the Raw Materials Committee, Council of National Defense, 1917-18. He was also appointed Director of Building Materials and of the Building Materials Division of the War Industries Board, which was created during the summer of 1918, and handled all matters relating to the conservation and distribution of building materials and supplies. He further served on the United States Highway Council, representing the War Industries Board in 1918. After the dissolution of the Board at the close of 1918 he continued to act for the Government in planning for the disposal of surplus building materials in such a way as to give the maximum price return to the Government and, at the same time, not unduly disturb the building materials market.

He was deeply interested in the welfare of the University of Pennsylvania. As President of the Engineering Alumni Society and as a Director of the General Alumni Society, he gave of his time and ability without stint. His

duties were discharged with courage and fidelity, and he has left his impress on the affairs of this great educational institution.

Mr. Humphrey was an outstanding character. His activities were numerous, his energy great. He achieved much in the field of engineering, especially in such organizations as the American Society of Civil Engineers, the American Society for Testing Materials, and the American Concrete Institute.

In the American Society of Civil Engineers he was a leader in the movement to establish Local Sections and in having them take an important part in Society activities. The developments which followed created a new era in Society affairs and resulted in greater service being rendered to the membership throughout the country.

He was deeply interested in the registration of engineers and served as Chairman of a Committee of the Society to draft a uniform law on this subject to be adopted in all parts of the United States. Such a law having been enacted in Pennsylvania, Mr. Humphrey was appointed by the Governor of the State, as Chairman of the Board to administer the law, which position he held from the institution of this Board in 1921 until his death. He was largely instrumental in establishing the highest ideals for the Engineering Profession in the operations of the Board.

The high regard with which the members of the Board held Mr. Humphrey may be best expressed by the words which they addressed to him on December 19, 1924, as follows:

"For a leading and distinguished engineer to have made the sacrifice of time and energy in the service of the State, that you have made, affords a notable example to the members of the Engineering Profession of how one of their number can lay aside personal interests, and forego large private gain, to give to the State high service for the promotion of the public good and the elevation of the profession of engineering."

In January, 1928, Mr. Humphrey was appointed a member of the Special Committee of the Board of Direction of the Society on Technical Expansion. On this Committee, as well as on all committees on which he served, he always gave unselfishly of his best thought in advancing the interests of the Society.

In the American Society for Testing Materials and in the American Concrete Institute, he had a most important part in the development of cement and concrete specifications and was recognized as an authority on these subjects.

Mr. Humphrey received a Gold Medal as collaborator at the Louisiana Purchase Exposition, St. Louis, Mo., in 1904, and the Medal of the British Concrete Institute in 1911. He was United States Delegate and Honorary President of the International Congress for Testing Materials at Copenhagen, Denmark, in 1909. He was appointed by President Roosevelt as a Member of the National Advisory Board on Fuel and Structural Materials and was its Secretary from 1905 to 1910.

He was a member of the Institution of Civil Engineers (Great Britain); Société des Ingénieurs Civils de France (Vice-President, American Section); British Fire Prevention Committee; National Fire Prevention Association; a Fellow of the American Association for the Advancement of Science; a member of the National Geographic Society; American Chemical Society; Society

of Chemical Industry (British); Société Industrielle Chimie; International Association for Testing Materials (Chairman of the International Committee on Constancy of Volume of Cement); American Society for Testing Materials (organized it and was its first Secretary, 1898-1900); American Railway Engineering Association; National Association of Cement Users (First President, 1905-13); American Concrete Institute (First President, 1913-15; First Honorary Member, 1926); awarded the Wason Medal in 1925 for two papers, "Twenty Years of Concrete" and "The Promise of Future Development"; Oesterreichischer Betón Verein; American Concrete Pipe Association; Secretary of the Joint Committee on Concrete and Reinforced Concrete, the constituent organizations of which were the National Societies of Civil Engineers, Testing Materials, American Railway Engineering Association, the American Concrete Institute, and the Portland Cement Association; Chairman of the Joint Committee on Specifications for Concrete and Reinforced Concrete (since 1920); Secretary of the Committee on Uniform Tests for Cement; Secretary of the Committee on Concrete of the American Society for Testing Materials; and a member of the Executive Committee of the Joint Conference Committee on Uniform Methods of Tests and Standard Specifications for Cement, consisting of representatives of the American Society of Civil Engineers, American Society for Testing Materials, and the United States Government. He was also Chairman of the governing body of American Engineering Council and Chairman of the Committee on Procedure, representing those National Engineering Societies which created the Federated American Engineering Society.

Mr. Humphrey was a man of remarkable ability and of comprehensive good judgment and rendered many useful services in the engineering world. He was especially helpful in his contacts with young men entering the profession and was active in organizing Student Chapters in the Society.

He was an indefatigable worker, thorough and painstaking in every thing he undertook, always an earnest and forceful advocate of the measures which he believed to be for the best interest of the Engineering Profession.

He was a staunch friend, a charming and lovable companion, and it was a privilege and an inspiration to be associated with him. The loss of his influence and counsel will be keenly felt at future meetings of the Technical Societies of which he was a member.

He is survived by his widow, Anna Kay Humphrey, and two sons.

Mr. Humphrey was elected an Associate Member of the American Society of Civil Engineers on May 5, 1897, and a Member on May 3, 1904. He served as Director from 1921 to 1923 and as Vice-President in 1925 and 1926.

WILLARD KENT, M. Am. Soc. C. E.*

DIED DECEMBER 22, 1924.

Willard Kent was born on October 27, 1851, at Marshfield, Mass., the son of Warren and Eliza W. (Weston) Kent. His ancestry extends back through various family branches to ten or twelve of the *Mayflower* Pilgrims.

* Memoir prepared by J. Parker Snow, M. Am. Soc. C. E.

His early life was spent in Marshfield where his father was a Building Contractor and where his primary education was commenced in the public schools. He afterward attended academies at Hanover, Mass., and Woonsocket, R. I.

From 1875 to 1884 Mr. Kent was employed as Chief Engineer in the office of the late John W. Ellis, M. Am. Soc. C. E., Consulting Engineer, of Woonsocket, during which time he managed the engineering work for the Providence and Worcester Railroad Company (now a part of the New York, New Haven, and Hartford Railroad Company), including the design and erection of repair shops at Valley Falls, R. I., as well as several passenger stations.

He was in charge of the design and building of the Woonsocket Water-Works, during 1883 and 1884, on the completion of which he was made Superintendent. In 1884, he opened offices in Woonsocket and Narragansett Pier, R. I., for private practice as a Consulting Engineer and Architect. He remained in charge of the Woonsocket Water-Works, however, until 1890.

Mr. Kent served as Consulting Engineer to the Woonsocket Electric Machine and Power Company, one of the very early municipal electric power plants of New England, the Nourse Mill of Woonsocket being the first textile mill in the United States to be electrically illuminated. He was in charge of the design, construction, and initial operation of the electric light plant at Woonsocket and also of the first electric street car lines in New England—a double trolley system put into operation about 1887.

Mr. Kent was married in 1876 to Jennie Moorhead, of Marshfield, who died in 1893, leaving two sons and a daughter. Later, he was married to Charlotte Angel, of Greystone and Narragansett Pier, R. I., who survives him. He maintained his legal residence in Woonsocket, although he lived principally at Narragansett Pier from 1893 until his death from cancer on December 22, 1924.

He was a member of various Masonic bodies in Rhode Island, as well as of the Boston Society of Civil Engineers and of the New England Water Works Association, having served as Secretary of the latter for many years.

Mr. Kent was elected a Member of the American Society of Civil Engineers on October 2, 1907.

JOHN HENRY STAATS, M. Am. Soc. C. E.*

DIED JULY 6, 1928.

John Henry Staats, the son of John Henry Staats and Jennie (Mapes) Staats, was born in New York, N. Y., on November 18, 1855. His ancestors on his father's side were among the first families who settled in Albany, N. Y., having come from Holland in 1629. On his maternal side he was descended from the Mapes family who came to the United States before the Revolutionary War, and made its home in Coxsackie, N. Y.

* Memoir prepared by Gerald Staats Rinehart, Assoc. M. Am. Soc. C. E.

Mr. Staats received his education in the public schools of New York City, and at City College. In 1872, he entered the office of Gen. Charles K. Graham to take up the study and practice of civil engineering.

In 1876 he was employed in the Department of Docks of New York City under General Graham, who had succeeded Gen. George B. McClellan as Chief Engineer of that Department. Mr. Staats served in the Department until 1882, at which time he had risen to the position of Assistant Engineer of Construction.

From 1882 to 1884 he was connected with the firm of Ross and Sanford, of Jersey City, N. J., which was engaged in river and harbor work in New York City and vicinity.

In 1888, with his brother, the late Robert Parker Staats, M. Am. Soc. C. E., he formed the partnership of R. P. and J. H. Staats which, later, was incorporated, and of which Mr. Staats served as Vice-President. During the time of its existence this firm constructed many notable works in New York Harbor, among which might be mentioned the North German Lloyd Docks in Hoboken, N. J., which were afterward used for the embarkation of troops during the World War, the Holland-American Line Docks, also in Hoboken, and three of the so-called Chelsea Piers in New York City below West Twenty-third Street. Mr. Staats was considered an expert on harbor work and dock construction and was retained by several of the transatlantic steamship companies in a consulting capacity.

In 1915, the Company was sold to the Snare and Triest Company of New York and Mr. Staats became Vice-President and Consulting Engineer of the latter firm. This position he held until his retirement from business in 1917.

He died suddenly at York Harbor, Me., on July 6, 1928, and is survived by his widow, Edythe Tillinghast Staats, and two daughters, Mildred Staats Nelson and Fredericka Staats Tuttle. He was a member of the Union League Club of New York, the Holland Society, and the Sons of the Revolution.

Mr. Staats was elected a Member of the American Society of Civil Engineers on March 5, 1884.

CADY STALEY, M. Am. Soc. C. E.*

DIED JUNE 27, 1928.

Cady Staley was born at Scotch Bush, N. Y., on December 12, 1840, the son of Harmanus and Evaline (Darrow) Staley. He worked on his father's farm and attended a district school and Jonesville Academy until he was sixteen years old, when he entered the Classical Institute, in Schenectady, N. Y. He went from there to Union College, where he took the course in Civil Engineering under Professor Gillespie. He was graduated with honors in 1865

* Memoir prepared by Frank H. Neff, M. Am. Soc. C. E.

with the degree of Bachelor of Arts, and, in 1866, he was given the degree of Civil Engineer.

As soon as Mr. Staley was graduated from college, he crossed the plains with a wagon train as the driver of an ox team, and after part of a year spent in prospecting for gold, he was employed by the Central Pacific Railway Company as a Civil Engineer. As such he was engaged on tunnel construction during 1866 and 1867, after which he returned to Union College as an Instructor in Civil Engineering. In 1868, on the death of Professor Gillespie, under whom he had served, he was made Professor of Civil Engineering.

Professor Staley, in 1876, became Dean of the Faculty, which position he held until 1886 when he was called to Case School of Applied Science, Cleveland, Ohio, as Professor of Civil Engineering and President of the Faculty.

As a result of his years of experience at Union College, Dr. Staley brought to Case well-defined ideas as to the organization and administration of an Engineering School. Requiring that he have full charge of the work done in the school, and ever willing to assume responsibility therefor, he, nevertheless, allowed the head of each department full power in the administration of that department, looking to him for results only. The success of Case School has proved the soundness of Dr. Staley's judgment, and what it is to-day is a monument to him as an organizer and administrator.

During the time that he taught at Union College and during the first years at Case School of Applied Science he was active in the practice of his profession, devoting his time to Sanitary Engineering. Dr. Staley early recognized the merits of the separate system of sewerage, and was one of the pioneers in promoting its adoption. In this work he had associated with him, George S. Pierson, M. Am. Soc. C. E., and together they designed and superintended the construction of separate systems of sewerage in West Troy, and Schenectady, N. Y., and in Dayton, Ohio.

Due to the lack of suitable texts on stresses in framed structures, Dr. Staley privately printed, during his early years at Union, the following booklets: "Notes on Bridge Engineering" (1875); "Strength of Materials and Stability of Structures" (1876); and "Elements of Truss Bridges" (1878). During this time he also revised and republished Gillespie's "Surveying" and "Roads and Railroads", and he, together with Mr. Pierson, published "The Separate System of Sewerage". After his retirement in 1902, he edited "The Teaching of Jesus".

In 1886, Union College conferred on him the degree of Doctor of Philosophy, and, in 1888, he received the degree of Doctor of Laws from both Union College and Ohio Wesleyan University. In 1920, Union College conferred on him the degree of Doctor of Science.

Dr. Staley was married at Waterford, N. Y., on December 29, 1869, to Kate Holcomb, daughter of Elvin and Philetta (Hall) Holcomb.

After coming to Case School Dr. and Mrs. Staley traveled abroad a great deal during their summer vacations. He brought back from these trips a wealth of architectural material which he gave to his classes in a series of illustrated lectures. These lectures were also open to the general public and were largely attended.

Dr. Staley retired in 1902, in order that he and Mrs. Staley might have ample leisure to travel, for which both had a great liking. After a trip around the world, they made their home either in Florida, or in Amsterdam, N. Y., where he had purchased a farm at Minaville, near the place where he was born. They continued, however, in their foreign travels.

Dr. Staley was a great lover of the horse and during his earlier years at Case he planned to retire upon a stock farm which he was starting in the West. He had imported Percheron stock from France, and looked forward with pleasant anticipation to the raising of heavy draft horses. However, a series of bad years caused him to abandon this plan and dispose of his stock and farm.

As a teacher, Dr. Staley had the faculty of presenting the principles of the comparatively simple science that Civil Engineering was in his early teaching days, in a way that fixed them in the minds of his students. In his duties as President, he ever had the welfare of the individual student uppermost in his heart. Whenever the application of a rule, well-designed for the good of the student body, might injure the future of an individual boy, he never hesitated to set it aside; and many a student of slow development, or whose preparation had been poor, owed his opportunity to start an engineering education, or his chance to make good after having started, to this quality of Dr. Staley.

Although retaining the title of Professor of Civil Engineering at Case for a number of years, Dr. Staley early turned over the teaching to those who were to succeed him, so that comparatively few had the privilege of working under him in engineering classes. Those who did, cherish the memory of the intimate personal relationship of the classroom, his lack of formality, and his sympathetic attitude toward their work. Although few had the classroom contact, there is no former student, nor member of the Faculty, who was at Case during the sixteen years that Dr. Staley directed its affairs, who does not hold his memory in highest esteem.

Dr. Staley was elected a Member of the American Society of Civil Engineers on March 17, 1869.

JOSEPH FRANCIS SINNOTT DONNELLY, Assoc. M. Am. Soc. C. E.*

DIED MARCH 5, 1926.

Joseph Francis Sinnott Donnelly was born in Philadelphia, Pa., on March 25, 1884. He was the sixth son of James A. Donnelly and Marguerite A. (Pope) Donnelly, both of whom were born in Philadelphia.

After graduating from the Central High School in 1902, he entered the Towne Scientific School of the University of Pennsylvania, receiving the degrees of Bachelor of Science and Master of Science in 1906 and 1907, respectively. While at College, he was a member of the Plumb Bob, the Civil Engi-

* Memoir prepared by R. F. Patterson, Esq., Camaguey, Cuba.

neering Society, and the Newman Club, and, after his graduation, he became affiliated with the General Alumni and Civil Engineering Associations, and the University of Pennsylvania Club of New York.

During the summer of 1906 Mr. Donnelly was employed as Inspector by the American Dredging Company, of Philadelphia. On the completion of his college work in 1907 he entered the employ of the American Bridge Company at its plant in Elmira, N. Y., as he had decided that structural engineering offered the most compatible field to him. In Elmira, he went through the preliminary stages of office work on structural design—tracing, detailing, etc.—remaining there until early in 1909, when he left to work on layout plans for the American Heating and Ventilator Company, of Philadelphia.

Later, in the same year, Mr. Donnelly accepted a position with the New York Central and Hudson River Railroad Company as Draftsman in the Office of the Engineer of Structures, Exterior Lines. He remained in this position about four years, detailing, designing, and checking plans of railway structures of various kinds.

In 1913, he accepted a position with the Cuba Railroad Company at Camaguey, Cuba, becoming, in turn, Chief Draftsman, Bridge and Building Engineer, and Office Engineer, which last named position he held at the time of his death.

The greater part of the thirteen years during which he was associated with the Cuba Railroad Company (which later became the Consolidated Railways of Cuba) was spent in Cuba. He also served for a time in the New York Office of the Company checking plans of structures for a new branch line and for new shop buildings. He was active in military service during the World War and was commissioned a First Lieutenant of Engineers, doing training and camp service.

During the time spent in Cuba, Mr. Donnelly was placed in charge of many important works of varied character. Extensions and branch lines were built, requiring new structures; lines were acquired, which also necessitated new, remodeled, and strengthened structures; and consolidation with the Cuba Northern Railway added greatly to the duties and responsibilities of his position.

Mr. Donnelly filled his several positions with the Cuba Railroad Company and elsewhere with marked ability which was attested by his steady advancement under each employer. His never-failing cheerfulness and tact, and his willingness to help other departments, as well as branches of his own, were highly appreciated by his employers and fellow workers alike.

He was a member of the American Railway Engineering Association and an organizer and Charter Member of the Knights of Columbus, of Camäguey, Cuba.

On December 27, 1919, Mr. Donnelly was married to Meta Jean Apeldorn, of Philadelphia, who, with two children, Ida Marguerite and Joseph Apeldorn, survives him.

Mr. Donnelly was elected an Associate Member of the American Society of Civil Engineers on August 31, 1915.

JOHN MARTYNN GRENFELL EDWARDS, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 24, 1928.

John Martynn Grenfell Edwards, the son of John and Lydia Edwards, was born in the small Cathedral City of Truro, Cornwall, England, on November 8, 1882. His father was an accountant, journalist, and artist, and was well known throughout the County of Cornwall for his work in connection with the Royal Cornwall Show. He was also a near relative of John Passmore Edwards, the well-known English journalist, newspaper proprietor, and philanthropist.

Mr. Edwards was one of seven children, he being the fifth. His father died at the age of forty-five when Mr. Edwards was ten years old. He received his elementary education privately, later attending the Truro Grammar School and the Redruth School of Mines. This was followed by a post-graduate course in Coal Mining in Wales where he won several prizes and a free scholarship for three years at the schools.

Before the completion of the mining course, the Boer War broke out, and he became a Trooper in Paget's Horse attached to the Imperial Yeomanry. He was seventeen at this time, and he received a four-bar medal, inscribed, "Cape Colony, Orange Free State, South Africa 1901—South Africa 1902".

On his return from the Boer War he found that a calamity had overtaken his family. His mother had befriended a poor woman who was dying from tuberculosis of the throat. Through the consequent infection, an only brother and three sisters died within a short period.

He resumed his studies at the mining school in Wales, and, in 1905, he accepted a position in India as Metallurgist with the Mysore Gold Mining Company. In 1908 he returned to England and worked as a Surveyor in a tin mine for the East Pool Mining Company, at Cornwall. A year later he went to the Rand, South Africa, to act as Surveyor for an underground development of the City Deep Gold Mining Company.

Mr. Edwards was engaged by the Messina Development Company, Limited, from 1910 to 1913 as Mine Surveyor, Chemist, and General Engineer in the development of a large high-grade copper mine at Messina, North Transvaal.

In 1913, he went to the West Indies as Superintendent for the Curacao Phosphate Company. He had charge of the installation of self-acting inclines and aerial ropeways, the construction of bins and wharves for shipping phosphates, the erection of buildings, machine shops, employers' houses, and native quarters, as well as the dredging of a channel from the sea to a lagoon which formed a natural harbor.

During the period of the World War, from 1915 to 1919, he accepted a commission as Lieutenant and, later, as Captain of a Tunneling Company in the Royal Engineers on the Somme. At different times on special work, he had charge of from 800 to 1 000 sappers. In the capture of Mont St. Quentin he was favorably mentioned in military dispatches by his superiors.

* Memoir prepared by L. A. Jenny, M. Am. Soc. C. E., and J. I. Leonard, Assoc. M. Am. Soc. C. E.

After the war, during 1919 and 1920, Mr. Edwards was associated with Professor Bernard W. Holman, of the Royal School of Mines, on research work in connection with the Cornish tin tailings. During this period, he did some prospecting in Spain on his own account.

The roving spirit still claiming him, he returned to Africa. For about a year he was Construction Engineer on some manganese property for the Dagwin Concession in West Africa, and for another year he was Irrigation Engineer for the Compagnie Générale des Colonies at Haut Senegal, West Africa. This Company was employed by the French Government to study the possibilities of the irrigation of the Niger Valley for the cultivation of cotton.

In 1924, Mr. Edwards went to South America, where he was employed as a Mining Engineer with the Poderosa Mining Company, in Chile, a concern engaged in the mining of copper. During the absence of the Manager for six months, Mr. Edwards was in complete charge of the mine. Remaining in South America, he later accepted an engagement with the Chile Exploration Mining Company supervising the construction of buildings and complete facilities for mining operations. Subsequently, during 1927, he worked on a hydro-electric development in Peru.

Early in 1928, Mr. Edwards went to New York, N. Y., and soon thereafter became associated with the valuation of the Interborough Rapid Transit Company. He was engaged on this work at the time of his death, on October 24, 1928, following an operation for appendicitis. He was buried with military honors in St. Michael's Cemetery, at Astoria, N. Y.

In 1919, he was married to Raymonde Helen Marie Louise Marguerite Vogues, daughter of M. Bertin Vogues, of Paris, France. M. Vogues was at one time Charge d'Affaires in Chile and Minister of Instruction in Paris, as well as an editor of prominence and a proprietor of scientific journals in France.

Mr. Edwards was a capable engineer and held in high esteem by the members of his profession. Those with whom he came in contact were invariably impressed with his kindly and courteous bearing and his character as a gentleman. To know him was to respect him and increasingly admire his many virtues. He was a good friend, a faithful brother, and a dutiful son. He is survived by two sisters living at Carbis Bay, Cornwall, England.

Mr. Edwards was elected an Associate Member of the American Society of Civil Engineers on January 16, 1928.

JAMES MADISON PORTER 3d, Assoc. M. Am. Soc. C. E.*

DIED JUNE 3, 1928.

James Madison Porter, 3d, the son of James Madison Porter, Jr., and Ruth Pierson (Cook) Porter, was born in Easton, Pa., on May 10, 1864.

He was descended from a long line of distinguished ancestors and his family has been prominent in the making of Pennsylvania history. His father,

* Memoir prepared by Lynn Perry, M. Am. Soc. C. E.

James Madison Porter, Jr., an eminent lawyer and distinguished diplomat at the Courts of Vienna and Berlin, was a Thirty-third Degree Free Mason and Grand Master of the Pennsylvania Grand Masonic Lodge. His grandfather, James Madison Porter, was for many years one of the leaders of the Pennsylvania Bar and was President Judge of the 23d Judicial Circuit which comprised at that time the Counties of Wayne, Pike, Monroe, and Carbon. Judge Porter was appointed Secretary of War by President John Tyler, but the appointment was not concurred in by the United States Senate. He was one of the founders and the first President of the Board of Trustees of Lafayette College, at Easton, a position which he held for twenty-five years, serving as Lecturer on Jurisprudence and Political Economy without compensation during the entire period.

Judge Porter's father, the great-grandfather of James Madison Porter, 3d, was Maj.-Gen. Andrew Porter, Commissary General of the American Forces during the Revolutionary War. He was offered the post of Secretary of War in the Cabinet of President James Monroe, but owing to the infirmities of age, declined to serve. General Porter had six sons, all of whom became prominent in National life: David Rittenhouse was Governor of Pennsylvania; George Bryan was Governor of Michigan; William Augustus Porter, LL.D., the son of David Rittenhouse, was District Attorney for the City of Philadelphia, Pa., and afterward Judge of the Court of the Alabama Claims; and another son of David Rittenhouse was Maj.-Gen. Horace Porter of Civil War fame. Rear-Admiral David Porter, U. S. N., Gen. Fitz-John Porter, and Gen. Andrew Porter were all collateral relatives of James Madison Porter, 3d.

Mr. Porter attended the public schools of Easton until 1879, when, following the death of his father, he, with his mother and sister, moved to Hackettstown, N. J. He prepared for college at the Centenary Collegiate Institute of Hackettstown and Blair Academy at Blairstown, N. J., and entered Lafayette College, in the autumn of 1882. When he went to Easton, he lived with his grandfather in the house that continued to be his residence until the time of his death.

In June, 1886, he was graduated from Lafayette College with the degree of Civil Engineer. Following short engagements with Davis and Grier and the Lehigh Valley Railroad Company, he entered the service of Tippett and Wood, Phillipsburg, N. J., as Chief Engineer. His duties in this position covered the design and construction of tanks and stand-pipes. He retained this connection for the remainder of his active career, even after he took up his college duties. During this time he was engaged on the design and construction of many steel structures. One of the most important of these was the cantilever bridge connecting Easton and Phillipsburg, referred to in many articles and books on bridge engineering. This bridge which will remain as a monument to his genius for many years, was built in 1895, and has no top chord bracing. Mr. Porter was well-known as a consulting structural engineer and, during his active career, he designed many highway and trolley bridges as well as many stand-pipes and elevated tanks. He was a pioneer in the development of pressure filters.

In 1890, on the death of Professor Joseph G. Fox, Head of the Civil Engineering Department at Lafayette College, Mr. Porter was appointed Instructor in that Department and became Adjunct Professor of Civil Engineering on June 25, of the same year. On October 22, 1891, he was made Professor of Civil and Topographical Engineering, in charge of the Department, and, in 1908, was appointed Professor of, and Director of the Division of, Civil Engineering. He occupied this chair until 1916 when he resigned after completing twenty-six years of service to his Alma Mater. Many generations of Lafayette men remember his outstanding qualities of leadership, teaching ability, sterling character, and genial disposition.

Professor Porter was active in church as well as technical, business, and civic affairs, and was a leader in every undertaking with which he was connected. He was a trustworthy friend of his colleagues and respected and beloved by all who were privileged to know him and who came into contact with him.

He served as Vice-President and President of the General Crushed Stone Company; Secretary of the American Society for Testing Materials; Director of the Easton Trust Company; President and Director of the Easton Tau Chapter of Zeta Psi; Vice-President and Director of the Northampton Country Club and the Pomfret Club. He was a member of the University Clubs of New York, N. Y., and Philadelphia, Pa., and the Automobile Club of America.

He was married on November 15, 1888, to Mary Virginia Drake, the daughter of Samuel Arndt Drake, of Easton, who, with an only son, James Madison Porter, IV, survives him.

Professor Porter was elected an Associate Member of the American Society of Civil Engineers on May 4, 1892.

HOWARD ELLIOTT, Affiliate, Am. Soc. C. E.*

DIED JULY 8, 1928.

Howard Elliott was born in New York, N. Y. on December 6, 1860, the son of Charles Wyllys Elliott and Mary (Whyte) Elliott. Charles Elliott Perkins, who as President so efficiently guided the destinies of the Chicago, Burlington and Quincy Railroad Company during its period of greatest expansion, was his cousin.

Mr. Elliott's early inclinations are shown in the fact that after passing through the High School of Cambridge, Mass., he entered what was then known as the Lawrence Scientific School of Harvard University, from which he was graduated with the degree of Civil Engineer in 1881. He plunged at once into railroad work, and, in October of that year, he entered the employ of the Burlington Railroad Company, with which he was to be intimately connected, either directly or indirectly, for many years. As a matter of fact he had been a Rodman on that road the year before he received his degree.

* Memoir prepared by Ralph Budd, and W. L. Darling, Members Am. Soc. C. E.

While he served in various capacities on various lines during his lifetime, Mr. Elliott's specific service as related to engineering may be said to have begun in 1896, when for five years he was General Manager of the Missouri Lines of the Burlington Railroad. In 1901, he became Second Vice-President of that Company, from which position, in 1906, he was promoted to the Presidency of the Northern Pacific Railway Company. He made his home at this time in St. Paul, Minn. From 1903 to 1913 there was a very marked development in the Northwest. During his administration as President he directed the greatest construction program ever attempted by the Northern Pacific Railway. The capacity of the railway was greatly increased, and the agricultural districts were better served by the construction of branch lines, and, in addition, the West Coast District was improved between Portland, Ore., and the Canadian boundary.

The late James J. Hill, F. Am. Soc. C. E., was then the dominant figure in the Northwest, especially in the group of railroads comprising the Great Northern, Northern Pacific, and Burlington Companies. The late E. H. Harriman was the leader in the Union Pacific and Southern Pacific group. It was Mr. Elliott's good fortune to be associated with these remarkable men and to take part in the memorable "Hill-Harriman" contests in railroad building in Washington, Idaho, and Oregon. This was a most important period in the history of the Northern transcontinental lines. While some remnants of the old feeling of bitterness remained, the community of ownership which had come about as a result in the formation of the Northern Securities Company was established more strongly day by day. Thus, the Great Northern and Northern Pacific Systems, growing more closely together, presented a united front to the rather insistent demands of the Harriman System. As part of the complete fortification, in a railroad sense, of the Pacific Northwest, the Spokane, Portland and Seattle Railway was constructed from Spokane, Wash., to Portland, a distance of 380 miles. Mr. Elliott had his part in this joint project, the conception and responsibility for which, of course, belonged directly to Mr. Hill.

The Northern Securities Company was declared illegal, under the Sherman Anti-Trust Law, upon suit by the United States Government; but the vital feature of this decision was that it did not affect the joint ownership established by the two Northern Lines of the Burlington property, its stock having been purchased outright and bonds issued to finance the purchase. At this time there was a deep hostility throughout the United States to railroad policies, and this enmity had to be turned first into neutrality and then into friendship. To succeed in this plan it was necessary systematically to carry on public education in the fundamentals of the situation. As a result Mr. Elliott was an important factor in such work through all his active life. He traveled across the Continent many times and made scores of addresses (some of which were printed), in which facts about the railroads were dispassionately presented before commercial clubs, business men's associations, and other bodies. The more formal talks which were distributed widely in the Northwest no doubt had a wholesome effect in the territory served by

the Northern Pacific System, and were very helpful in directing the attention of the people to the need of remedial legislation.

By the summer of 1913 there had developed in New England a railroad situation as unpromising and as difficult to handle as that which had existed in the territory of the two Northern transcontinentals. The New York, New Haven and Hartford Railroad Company was the great railroad corporation of New England. It had been expanded to take in most of the railroad properties of that section, and its finances had become seriously affected. Much hostility had been aroused against the management of the road, particularly over the policy which included a consolidation of all New England transportation activities—steam railroads, trolleys, and steamship companies—to be dominated by the New Haven Company. A number of prominent New Englanders interested in an adequate and permanent transportation service for New England, chief among them Mr. Elliott's friends, the late Henry Lee Higginson and Theodore N. Vail, invited and urged Mr. Elliott to come East and take up the task. As his children were either at school or college in the East, and practically all his relatives and old friends were there, he found the idea attractive. Then, too, since the Board of Directors of the Northern Pacific Company met in New York, it had been necessary for him to make from eight to ten trips a year between St. Paul, Minn., and the Atlantic Coast, which, with the necessary inspection trips in the West as far as the Pacific Coast, represented traveling from 30 000 to 40 000 miles per year.

Mr. Elliott, realizing to some extent the seriousness of the situation, decided to take up the work, after discussing the matter fully with friends in Boston, Mass., New York, and Philadelphia, Pa. Making his residence in Boston, he became associated with the New York, New Haven and Hartford Railroad Company as its President on September 1, 1913, and soon became Chairman of the Board of Directors, being succeeded by Mr. James H. Hustis as President. At that time the New Haven System consisted not only of its rail lines, but also included in its ownership or control the Central New England Railway Company, the Boston and Maine Railroad Company, the New York, Ontario, and Western Railway Company, the New England Steamship Company, the Hartford and New York Transportation Company, substantially all the street railway lines in Rhode Island and Connecticut, and the important street railways of Massachusetts, in Berkshire County, and in the Cities of Springfield and Worcester. It was expected that the Chairman of the System would exercise general oversight of the holdings of the New Haven Company, each of which was to be in charge of a President, and that the Board of Directors of the New Haven Lines, being responsible for the parent company and its holdings, should act as a supreme advisory body to the Chairman. These plans were soon upset by the action of the Federal Government, however, in bringing suit to compel the New Haven Company to give up the Boston and Maine Railroad, and the Massachusetts, Rhode Island, and Connecticut trolleys.

Mr. Elliott labored long and earnestly to avert the Consent Decree, which was forced on the New Haven Company by the Administration at a time when this Company, because of its financial embarrassment, could not afford to

resist the Government. Mr. Elliott accepted the situation, however, and with what remained of the New Haven System began earnestly to present to Southern New England the serious transportation problems which confronted the transportation agency and the communities which were dependent upon it. In his speeches and writings he urged the necessity for confidence, patience, and co-operation, and, greatly to his credit, secured them all. His attitude and action inspired a revival of the belief that the New Haven System might be restored to its former high state. Incessant work in this field impaired his health, and on May 1, 1917, he resigned as Chairman and President, and took a position on the Railroads' War Board, at Washington, D. C., where, together with the other members of the Board, he represented all the railroads until the beginning of Federal Control in the same year.

In 1920, Mr. Elliott became Chairman of the Board of Directors of the Northern Pacific Railway Company, and took an active part in the management of that property until his death. He was also a Director of the Chicago, Burlington and Quincy Railroad Company from 1918 until his death, as well as a Director of many companies connected with railroads and other allied interests, as well as a member of many clubs and scientific, business, and social organizations.

Mr. Elliott was also a very ardent and enthusiastic Harvard man, and Harvard University, in turn, thought most highly of him. As early as 1909 he was elected one of its Board of Overseers, and the Alumni Association made him its President for the year 1917. He was extremely interested in the work of the University, especially that of the School of Transportation, founded in honor of Mr. James J. Hill.

The honorary degree of Doctor of Laws was conferred on Mr. Elliott by several colleges, namely, Middlebury College, Vermont, in 1916; Trinity College, Connecticut, in 1914; and Carleton College, Minnesota, in 1927. He also received the honorary degree of Doctor of Science from the University of Pennsylvania in 1927.

Mr. Elliott was married on October 12, 1892, to Janet Churchill. They had two daughters and one son. He was an extremely attractive man personally because he seemed actually to radiate kindness and good will. These qualities contributed greatly to his practical success.

Mr. Elliott died on July 8, 1928, while still actively engaged in the handling of railroad affairs to which so much of his life had been devoted. Few men have left behind them a larger circle of appreciative associates and admiring friends.

Mr. Elliott was elected an Affiliate of the American Society of Civil Engineers on June 5, 1900.